Foundations

of

Bridges and Buildings

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of

Bridges and Buildings

BY

HENRY S. JACOBY

Professor Emeritus of Bridge Engineering
Cornell University

AND

ROLAND P. DAVIS

Dean, College of Engineering
West Virginia University

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PREFACE TO THE THIRD EDITION

The many developments in the field of foundation engineering since 1925 have necessitated revision in practically all chapters; hence the book has been completely reset. The development of the field of soil mechanics has brought a new importance to foundation exploration work. Consequently this chapter, which in the first and second editions was placed at the end of the book, has been rewritten and made the first chapter.

The field of soil mechanics is too extensive to permit of full treatment in a general text on foundations of bridges and buildings, but some of the more important phenomena in this important field are given in the second chapter.

The following new material on piling is presented: proper hammer weights, new formulas for bearing capacity, added information on marine borers, lateral strength of piles, steel H-section piling, and design of sheet-piling installations.

New material on cofferdams and caissons include deep cofferdam construction, design of cofferdams, placing cylinder caissons by boring, deep concrete open caissons, compressed-air flotation caissons, the sand-island method of placing caissons, air locks and rules for working in compressed air.

Material has been added to the text on grouting. The use of boring machines for placing deep cylinder piers is described, as is also the subject of predraining foundations. The article on the obstruction offered by bridge piers to the flow of water has been amplified and an article added in the offset method of underpinning. Many additional changes will be found in the form of new paragraphs here and there throughout the book.

Acknowledgment is made to the following for the use of material and of illustrations: *Engineering News-Record*, *Civil Engineering*, Boston Society of Civil Engineers, American Railway Engineering Association, American Society for Testing Materials, Union Iron Works, Bethlehem Steel Company, United States Steel Corporation, Industrial Brownhoist Company, Portland Cement Association, Keystone Driller Company, Raymond Concrete Pile

Company, C. B. McCullough, Arthur Casagrande, Lazarus White, H. A. Mohr, Philip C. Rutledge and M. Juul Hvorslev.

The revision work of this edition was done by the junior author. He accepts responsibility for all new material, as well as for the many changes in the arrangement of material that appeared in previous editions.

R. P. Davis.

Morgantown, W. Va.,
April, 1941.

PREFACE TO THE FIRST EDITION

In preparing this volume the aim of the authors has been to treat in a systematic manner the entire subject of foundations for bridges and buildings as represented by American engineering practice. Only occasional references are made to foreign practice. It was hoped, at first, to accomplish this task within the limits of about 300 pages, but, as the work progressed, it became evident that this could not be done without abbreviating the treatment of many topics so much as to become unsatisfactory. In many cases, space has been economized by inserting additional illustrations and reducing descriptions in the text.

A large proportion of space is devoted to piles and pile driving, since young engineers are more likely to obtain their early experience with pile foundations than with any other class of foundation construction. Many facts derived from experience are given to emphasize and illustrate the application of fundamental principles and to form a rational basis for that kind of judgment which is such an important element in an engineer's professional practice. The undesirable features of considerable pile driving in this country have been due as much to the assumption that the art of pile driving is so simple that the aid of science is not essential as to the attempt of some engineers to base the art upon theoretical rules which fail to take into account many practical factors of the problem. Another reason for extending the treatment is due to the recent introduction of concrete piles which will help to retain the dominant place that pile foundations have held heretofore among other classes of foundations.

The attention of engineering teachers is called to the arrangement of the topics in the first five chapters. Instead of combining the treatment of all kinds of piles in chapters on descriptions, equipment, driving, and bearing power, respectively, the subject is developed in accordance with pedagogical principles for the benefit of students who approach it without any previous knowledge of the subject. It is believed, however, that practicians will find this arrangement equally useful for their study and reference. The full

discussion of the bearing power of timber piles before considering that of concrete piles, conforms also to the order of historical development.

The treatment of the pneumatic process and its application, to both bridges and buildings, is supplemented by a chapter on pneumatic-caisson practice by T. Kennard Thomson, an experienced consulting engineer who has specialized in foundation construction. The results of his experience and observation should be helpful to all engineers and contractors of lesser experience.

Three chapters on piers and abutments are incorporated in this work since courses of instruction in technical colleges frequently include these topics in masonry construction with foundations. During the past decade considerable improvements have been made in the design of piers and abutments by the introduction of new types, including hollow and arched forms, in order to reduce the loads upon foundation beds and to eliminate a large part of the lateral thrust of embankments, as well as to decrease the volume of masonry in some cases.

The limits of the volume precluded historical notes in connection with every class of foundation, but they are introduced in certain cases relating to new types of construction, or where the process of development indicates the features which are likely to persist in the future.

Since a subject embracing so many details of design and construction cannot be exhaustively treated in a single volume of convenient size to meet the needs of all practicians, a chapter has been added which contains a large number of carefully selected and classified references to the vast amount of illustrative material on foundations contained in engineering periodicals and the proceedings of engineering societies. It is hoped that young technical graduates will form the habit of consulting the articles referred to, making suitable abstracts, and filing them for future use. To compare the manner in which different designers have solved a given problem is a most valuable study.

Grateful acknowledgments for photographs are due to S. W. Bowen, A. S. Crane, A. O. Cunningham, Dravo Contracting Co., Lackawanna Steel Co., Ralph Modjeski, C. K. Mohler, J. H. Prior, J. R. Rablin, E. J. Schneider, H. E. Stevens, F. L. Thompson, and M. M. Upson; to J. Q. Barlow, J. D. Isaacs, and H. K. Seltzer for permission to reproduce drawings; to R. A. Cummings, *Engineering News*, *Engineering Record*, *Engineering and Contracting*, and *Railway Age Gazette* for permission to reprint illustrations; to C. W. Reinhardt

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April 15, 1914.

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SOIL EXPLORATIONS AND BEARING CAPACITY

1-1. Foundations. A structure usually consists of two parts, one of which is supported by the other, the upper part being known as the superstructure and the lower part as the substructure. In a bridge the superstructure is composed of the beams, girders, and trusses, together with the floor system and bracing which they carry, whereas the substructure consists of the piers and abutments, including their supporting bases.

The substructure frequently consists of two parts which differ more or less in form and character, the lower part being called the foundation, this supporting the rest of the structure. Sometimes the term "foundation" is used without regard to any substructure, as, for example, when it is applied to the independent structure which supports a machine.

The foundation of a structure may then be defined as that part of the structure which is usually placed below the surface of the ground and which distributes the load upon the earth beneath.

Foundations are divided into various classes. The simplest form is obtained by merely widening the base of a wall or pier, so as to distribute the load over sufficient area on the foundation bed of earth. Another form is known as a "spread footing," in which the bearing area of a wall or pier is enlarged either by reinforcing the concrete base with steel bars or by inserting one or more tiers of steel I-beams. Large buildings resting on poor bearing soil may have a spread or raft foundation in the form of a reinforced-concrete slab that covers the whole basement area.

Pile foundations consist of a base of concrete or of timber grillage, supported by piles which distribute the load to the earth through a considerable depth either by friction alone or by friction combined with bearing on the ends of the piles.

When the bottom of the foundation has to be located on a bed of firm material at a considerable depth below the surface of the ground, the classes of foundations are designated by the respective methods required to sink them into position.

Foundations built in open excavation, or in open wells, are used when the excavation can be made either in the dry or with no more interference by water than can be controlled by a reasonable amount of pumping.

Where open caissons are employed, the excavation is made through the water under ordinary atmospheric conditions; after the bottom is sealed by concrete, the rest of the foundation is built in the open air.

Pneumatic-caissons are those in which the excavation is made by working in compressed air in the chamber of the caisson, on the roof of which the concrete or masonry is built up in the open air during the process of sinking.

Many kinds of foundations also require the use of a temporary structure known as a "cofferdam," which excludes the water from the site of the foundation during its construction.

The kind of foundation to be adopted depends largely on the character of the soil at the site and also on the presence or absence of water. The above-noted general classes of foundations, and their subdivisions, are described and illustrated in the following chapters of this volume.

The science and art of foundation design and construction have lagged considerably behind the science and art of superstructure design and construction; and yet the difficulties encountered below the ground are much greater than those found above the ground level. The superstructure will be the same wherever built, but the substructure must be designed to fit the particular soil conditions obtaining at the site. Foundation failures are generally not due to structural defects within the substructure itself but rather to a yielding of the soil supporting the substructure. A moderate amount of uniform settlement may be permissible, but differential settlement—a varying settlement in different parts of the structure—may lead to serious consequences by producing excessive stresses in the structural elements and by causing unsightly cracking.

In studying any foundation problem, the first step should be an investigation of the soil conditions, in order (*a*) to provide the necessary data by which the engineer may determine the most economical type of substructure and its proportions and (*b*) to furnish the contractor with the necessary information for carrying

on construction work with maximum speed and economy. The investigation will include an exploration survey to determine the general nature and thicknesses of the several strata penetrated, as well as laboratory and field tests for bearing capacity determinations.

1-2. Need of Subsurface Explorations. Because of the general lack of proper investigation of subsurface conditions, underground work is still the biggest gamble in both engineering and construction. Adequate explorations are often omitted because of the time and cost involved. Innumerable examples demonstrate that this is false economy, for the cost of exploration is frequently less than the expense involved in merely revising the plans of the structure, without considering the unnecessary cost of the structure due to lack of proper information. Inadequate foundation investigations invariably result in greatly increased costs, and sometimes even in the loss of the structure itself.

In one instance a bridge pier was built on the surface of hardpan in a river bed. No examination was made on account of the swift current. Without warning the pier sank out of sight, causing the loss of two adjacent spans and a number of lives. On making an investigation afterward, it was found that the hardpan was only a thin stratum overlying a deep layer of soft clay.

In another example a bridge abutment which was founded on 60-ft. timber piles settled slowly until it reached a maximum of 3 ft. Exploration showed that the settlement was due to a 10-ft. layer of peat 35 ft. below the surface, which apparently was flowing under the superimposed load.

In placing the foundation for a building in New York City in which steel-cylinder piles (Art. 7-8) were used, a great deal of trouble was experienced because of the presence of buried stone-filled cribs. The actual conditions were not known previous to construction, as exploratory work was not permitted inside the existing building. The preliminary investigations were limited to a few core-drill holes through the sidewalk outside of the property lines.

Pile driving was started by using 12-in. pipe with shell thicknesses of $\frac{3}{8}$ and $\frac{1}{2}$ in., but with these thicknesses from 25 to 45 per cent of the piles were ruined in attempting to force the same through the cribs. Better results were obtained when the shell thicknesses were increased to $\frac{3}{4}$ and $\frac{7}{8}$ in., although in one spot the use of piles had to be abandoned, a timbered open pit being substituted.

The Washington Monument, designed for a height of 600 ft., was built on a deposit of good bearing material consisting of closely compacted sand and gravel, the base being 80 ft. square. Started

in 1848, construction reached a height of 150 ft. in 1854, when work ceased owing to a lack of funds. It was not resumed until 1880. In the meantime the structure settled until the axis was leaning $1\frac{3}{4}$ in. to the north.

Before starting work anew a committee of engineers was retained to examine the foundation. This committee recommended that the height of the proposed shaft should be decreased to 500 ft. and that the area and the depth of the footing should be greatly increased. These recommendations were largely carried out. By a skillful system of underpinning, the axis of the shaft was brought back into plumb. The footing was increased $13\frac{1}{2}$ ft. in depth and enlarged to $2\frac{1}{2}$ times the area of the old footing. Based on a total weight of 81,120 tons, the bearing load is 5 tons per sq. ft. However, the enlargement of the base did not eliminate continued settlement. When the work was completed about 1886, the settlement during construction amounted to approximately $4\frac{1}{2}$ in., which has since increased about 1 in.

The committee referred to above made borings penetrating 18 ft. into the sand and gravel. No reason is given in the records as to why the borings were not carried deeper, for tests made in 1932 showed that the site is underlaid with a blanket of soft clay over the rock, which is at elevation −60. This clay is responsible for the very considerable and continuing settlement.

In making subsurface explorations the two following requirements must be kept in mind: (*a*) The tests should be carried to rock or to an elevation that leaves no doubt as to the character of the material below and (*b*) the method used should give complete information as to the character of the soil and the degree of compaction and also as to water conditions.

Another important reason why adequate explorations should be made is that the owner ought to assume full responsibility for the local conditions and the contractor should not be obliged to gamble on uncertainties related thereto.

In general, two sets of borings should be made for an important bridge crossing. In the first set a number of borings are located on the center line of the proposed location to determine whether the site furnishes favorable conditions and, if so, to locate the proper position of piers and abutments. After these are tentatively located, additional borings should be made at the site of each pier and abutment. At least four borings, one at each corner, are usually desirable, and often several intermediate ones may be required to plat accurately the positions of the several soil strata.

In building construction one boring should be made at each corner of the building and as many more as are required to give complete information of the soil pattern.

1-3. Classification of Bearing Materials. Bearing materials may be divided into four classes: (*a*) rock and hardpan, (*b*) gravel and coarse sand, (*c*) fine sand, and (*d*) silt and clay. Where rock is available as a bearing material at a reasonable depth the foundation problem is solved, for the crushing strength of most rock exceeds that of the substructure masonry. Therefore, when the latter is properly designed, adequate bearing area on the rock bed will be provided. Hardpan also offers an unyielding support when the stresses are kept within proper limits, and it is classified as excellent bearing material.

Gravel and coarse sand are classed as good bearing materials where danger from scour is absent, although, of course, for equal loads the bearing area must be larger than for rock or hardpan.

The bearing characteristics of fine sand vary widely according to the water content, fine sand being quite stable under some conditions and extremely unstable—quicksand, for example—under other conditions.

Silt and clay, which are extremely fine grained soils, are classified as poor bearing materials; for, in general, any considerable load placed on the same results in a pronounced settlement. This settlement is due to a combination of (*a*) lateral flow, in which the plastic silt or clay is displaced from under the bearing area, and (*b*) vertical deformation due to compression. This compression deformation results from a consolidation or compaction of the soil grains by squeezing the water and air from the spaces between the grains. The loading is generally not large enough to cause crushing of the soil particles or any considerable elastic deformation of the soil grains themselves. Owing to the high impermeability of clays, the movement of the water, as it is forced out, is very slow; consequently, settlement is gradual but may continue for a long time.

In nature we are likely to find mixtures of silt and clay with sand and gravel in a wide range of proportions. Where enough sand or gravel is present to give a nonplastic soil, a stable and nonyielding foundation may be expected.

One of the most perplexing phenomena of a soil is the change resulting from a disturbance of its natural structure. In its undisturbed state it may be stable and capable of resisting considerable stress without undue deformation, but when its natural structure

is broken down it becomes weak. Hence, to secure a stable and reliable bearing, the natural structure should not be disturbed. A good bearing material may be ruined during excavation by allowing ground water, seeping from above, to run down on it, or by permitting men to churn up the surface by walking around on it.

In fine-grained soils the effect of a change in the evaporation of capillary water is a disturbing feature. At the surface of the ground the sun evaporates this capillary water as fast as it rises. Hence the moisture content will vary from zero at the surface to a maximum at ground-water level. Any structure erected will shield the ground beneath from the sun's rays and result in an increase in the moisture content near the surface. In certain yellow clays in Texas this moisture increase will cause a bulging up of the soil, which has been corrected in some instances by draining the soil with tile drains placed in trenches filled with large stones. This construction permits the entrance of air, as well as the drainage of the water near the surface. The air stops the capillary action.

These observations show the vital effect that changes in the moisture content have on the physical properties of fine-grained soils. Over a long period of time it seems to be characteristic of nature to so consolidate a soil as to give it adequate strength to carry the overburden. For example, a certain soil, 20 ft. below ground surface, carries a load of 2,300 lb. per sq. ft. due to the weight of the earth above the plane in question. Under this pressure it has reached a state of equilibrium as far as further consolidation or settlement is concerned. If this pressure is now increased by placing a structure on the site, further consolidation takes place by driving out more of the water in the soil, until equilibrium is again established. In sand, where resistance to flow is low, the adjustment will be rapid; but in impermeable clays it will be very slow.

In foundation design the problem is to determine how much weight can be placed on a soil in excess of that previously carried in the form of overburden. In the design of some structures, such as buildings with deep basements, the weight of the structure plus the live load may not exceed the weight of the soil excavated, in which case no settlement will be expected if a raft foundation is used covering the whole basement area.

In making foundation explorations the above-noted phenomena must be kept in mind. Where a bearing test is made, it should be on an undisturbed bed as near the elevation of the bottom of footings as possible. If soil samples are taken for laboratory analyses, care

must be observed to secure undisturbed samples, both as to the degree of consolidation and as to the water content.

1-4. Sounding Rods. Among the methods used in making foundation explorations are the use of sounding rods, earth augers, wash borings, test pits, and core drills.

A sounding rod consists of a steel rod or pipe, usually $\frac{3}{4}$ or 1 in. in diameter, put together in sections 4 or 5 ft. long by the use of outside couplings. These couplings are so threaded that the ends will butt together at or near the center of the couplings to relieve the threads of stresses during driving operations. The bottom section is pointed, and a drive cap is used on top. The rod is driven into the ground by means of a maul or sledge hammer. Another method of driving is by the use of a hollow cylindrical ram which encircles, and is guided by, the rod. The ram weighs about 50 lb. and is raised by hand. In falling it strikes on a driving head fastened to the sounding rod. Sounding rods have been driven to a depth of 85 ft. by this method.

The main function of a sounding rod is to determine the elevation of rock, although it may also show whether soil resistance is increasing or decreasing. A sunken log, boulder, or other obstruction can stop the driving. The information thus secured may be so inadequate that it is misleading. The rod may find a stratum of gravel but because of the high resistance may fail to reach soft clay beneath the gravel. In one instance eight men on the handle bar were unable to push the sounding rod over 7 ft. into stiff mud or clay, but, on driving test piles, no difficulty was found in driving 70-ft. piles. In another case rods drove easily for 60 ft. and three men lifted the full length of the rod out of the ground, yet a building is standing there today supported by 22-ft. piles.

Figures 1-4*a* and 1-4*b* show the details of a more elaborate sounding-rod outfit used for locating the elevation of rock to a maximum depth of 30 ft. along the line of the Welland Ship Canal. Where the drill rods could be withdrawn without much trouble, solid points were used, but for deep drilling the rods had removable points in the form of cast-iron sleeves, these sleeves remaining in the ground on withdrawal of the rod. In the illustration 2-in. rods are shown, but $1\frac{1}{2}$-in. rods proved just as satisfactory. The hammer weighed 125 lb.

The Ohio Department of Highways, cooperating with the Engineering Experiment Station of The Ohio State University, has developed an elaborate power-driven sounding-rod outfit, the use of which has made possible the predetermination of piling require-

ments in bridge building.[1] Calibration curves have been prepared to show the relationship between the resistance encountered in driving piles and that encountered in driving drill rods.

The outfit consists of a gasoline engine mounted on a four-wheel chassis, together with 13-ft. vertical leads and a 122-lb. drop-hammer. The hammer drives a pointed sectional rod made of standard 1-in. double extra-heavy pipe, having an outside diameter of 1.315

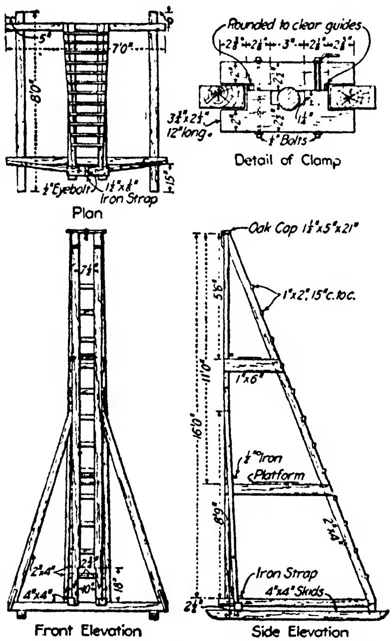


FIG. 1-4a.—Light Leads of Drill Outfit Built to Drive and Pull Sounding Rods.

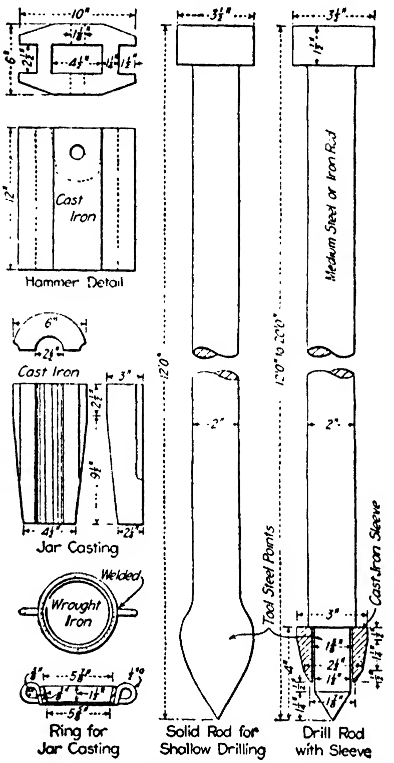


FIG. 1-4b.—Details of Drill Rods and Hammer for Light Sounding Outfit.

in. The upper end of this rod is guided and protected from the blows by a driving head and follower, the latter being fitted in the leads for guidance. After a drive has been completed, the test rod is pulled out of the ground by a hollow screw-power jack. This outfit is also designed to do churn-drill work where necessary.

A sounding-rod outfit was used in New York City subway work to determine the bearing capacity of sand by its resistance to pene-

[1] The Predetermination of Piling Requirements for Bridge Foundations, *Ohio State Univ. Eng. Exp. Sta. Bull.* 90, 1935.

tration. The apparatus consisted of a conical shoe about $2\frac{3}{4}$ in. in diameter, attached to double extra-heavy $1\frac{1}{2}$-in. pipe. To permit withdrawal of the shoe and also to eliminate all friction except at the lower end, the shoe was jacked down through a casing $3\frac{1}{2}$ in. in diameter. The ram of the jack was pushed down 2 in. at a time, the gage reading was noted, and the process was continued until the full 12-in. stroke was reached. Water was then turned in through the shoe, and the casing was forced down to the shoe level, after which jacking was resumed.

To calibrate the penetration figures, test pits were dug along the sides of several borings, and points for loading tests (Art. 1-16) were selected where the penetration tests showed changes in the density of the soil. The loading tests were made on 1-sq. ft. octagonal plates, the load being applied through hydraulic jacks.

1-5. Augers. A simple and effective tool for use in exploring shallow foundations in sand and clay consists of an ordinary wood auger, 2 in. in diameter, fitted into a section of 1-in. pipe, which in turn is connected by ordinary couplings to a series of pipe sections. As shown in Fig. 1-5*a*, a handle about 2 ft. long is provided at the top.

Fig. 1-5*a*.—Hand Auger.

In starting, care is necessary to keep the auger vertical. Five turns fill the bit, which is then withdrawn to the surface and cleaned after examining the material. Where the hole is too deep to raise the auger by hand, it is lifted by a block and fall suspended from a tripod, a chain being used to grip the pipe. If the hole becomes partially clogged, it may be necessary to reduce the size of the auger to 1-in. diameter, connected to sections of $\frac{1}{2}$-in. pipe. In dry sand it is necessary to pour enough water into the hole to make the grains stick together so that they may be lifted.

When sand or gravel becomes troublesome and the hole will not retain its shape, a 3-in. casing is driven, being handled in 4- or 5-ft. lengths, in which case the cost becomes much greater and the method approximates that described in the following article. Where boulders are encountered, a drill with a chisel point attached to sections of pipe may also be needed. Even with the aid of casing, the auger method may not be applicable in quicksand unless the layer is thin, when it may be removed with a sand pump. This

sand pump consists of a cylindrical bucket with a cutting edge at the bottom and above this a flap valve opening upward. It is partly filled by rapidly raising and dropping it alternately.

In addition to the regular wood auger, special types of augers have been developed. Figure 1-5*b* illustrates one type that will bore holes up to 12 in. in diameter. A regular posthole digger (Fig. 17-9*a*) may be used to depths of about 16 ft.

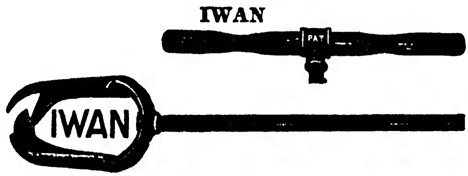


FIG. 1-5*b*.—Posthole Auger.

Another type of sampler used for soft materials at considerable depths is shown in Fig. 1-5*c*. It consists of a brass core barrel at the end of a $1\frac{1}{4}$-in. pipe, inside the core barrel there being a steel plunger at the end of a $\frac{1}{2}$-in. pipe. The instrument is forced down into the ground and successive lengths of $1\frac{1}{4}$- and $\frac{1}{2}$-in. pipe added until the desired depth is reached. The plunger is then drawn up

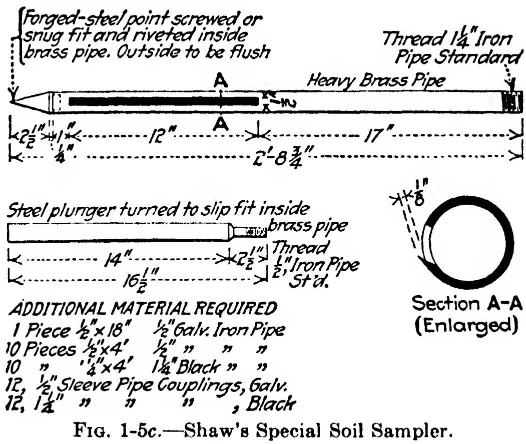


FIG. 1-5*c*.—Shaw's Special Soil Sampler.

about 2 ft. and the larger pipe rotated, the lip of the brass pipe cutting off a sample.

The borings from augers are regularly inspected as the ground is penetrated and a record kept of the depths and variations in materials encountered. Satisfactory information may be obtained as to the character of the soil but not as to its water content and degree of consolidation. Borings with augers are used for depths

up to 100 ft. The cost may be as low as 15 cts. per foot for shallow holes without casings, but where casings are used the cost may vary from 75 cts. to $2 per foot.

1-6. Wash Borings. Wash boring is the term used to designate the method of soil exploration in which a casing is driven and the soil inside washed out. This method can be used for almost all soils except hardpan or where boulders are present.

A standard outfit consists of a small derrick or tripod, the casing, drive weight, hollow drill rods, and a hand or power pump, together with their accessories and necessary tools. A convenient size of tripod has wood legs 3 or 4 in. square in section and 18 ft. long, or the legs may be made of pipe. The rope for raising and lowering the casing and drill rods is manipulated either directly by hand or with the aid of a drum.

The casing is composed of extra-heavy steel or iron pipe in about 5-ft. lengths. A number of types of standard couplings are available, but one of the most satisfactory is an outside malleable-steel recessed coupling. It is sturdy and will withstand driving and pulling; the recessed ends provide lateral stiffness. The usual size of casing has a nominal inside diameter of 2½ in.

The casing is driven with a weight operated as a drop-hammer (Fig. 1-11*c*). This drive weight rides a pipe which is threaded to screw on the casing and which acts as a guide to the weight. An anvil block is attached to this guide pipe to receive the hammer blow. Drive weights for hand operations usually weigh about 150 lb., while for power operations they may weigh as much as 800 lb. The casing is driven only as far as is necessary to keep the wall of the bore hole from caving. Where the soil is clean loose sand, the casing will be driven the full depth of the hole; but in silt, peat, or clay, or even in compact sand where water conditions are favorable, the casing may stop at a much higher elevation. A set of blocks and fall is employed to remove the casing on completion of boring. If the resistance is large, this force may be augmented by using the drive weight reversed.

Heavy black pipe, 1 in. in diameter and in 5-ft. sections, is used to wash out the soil in the casing. The threading and couplings are usually the same as for the casing. The bottom section consists of a hollow chopping bit where hard material is encountered. The top of the pipe is fitted with a water swivel for rotating the wash pipe. Various types of chopping bits are in use, such as the chisel-edge bit, the fishtail bit, and the X-shaped chisel bit. Their purpose is to cut the material loose by churning the wash pipe up and down

and twisting it at the bottom of the stroke. Water ports are placed in the sides of the bits.

Almost any type of hand or power pump may be used to circulate the wash water that will furnish from 20 to 60 gal. of water a minute at a pressure of 50 lb. per sq. in. City water pressure may be sufficient to do the work. The order of procedure is as follows: After driving a section of casing, the material inside is washed out by inserting the wash pipe in the casing and starting the pump. Water is usually taken out of a tub through a flexible hose and, after passing through the pump, is forced down through the wash pipe. It washes the soil up through the annular space between the casing and the wash pipe and out into the tub through a horizontal section of pipe connected near the top of the casing by a T. The tub serves as a storage basin to permit using the water over and over, as well as to reclaim the soil washed through the drill hole.

The operation of adding a section of casing, driving the casing 5 ft., adding a 5-ft. length to the wash pipe and then washing out the casing is repeated until the required depth has been reached or until obstacles, such as boulders, are encountered. If the boulder is not too large, a small charge of dynamite may be used by an experienced operator, care being taken to raise the casing high enough to avoid any damage from the explosion.

Information obtained from wash borings is not generally very satisfactory. Samples are taken by reaching the hand into the bottom of the tub or by placing a receptacle under the end of the discharge pipe. In either case only the coarser grains are picked up, for the finer grains are in suspension or have been washed away. It may also be difficult to determine from just what elevation any particular sample has come.

If the main purpose of the wash-boring method is to determine the elevation of bedrock, this may be difficult, for it is not always possible to distinguish between a large boulder and bedrock. By making a number of borings at different locations and comparing the elevations of the supposed rock surface, it may be assumed that, unless they are nearly at the same level or fit some fairly definite pattern, the more erratic elevations indicate boulders. Additional borings should then be made near these locations.

The cost of wash-boring explorations will vary between wide limits, depending on the difficulties encountered and on the size of the job. Where the casing can be used a number of times, the cost will be reduced. In general, wash borings will cost from 50 cts. to $1.25 per foot, but for large projects the cost may be much less.

1-7. Dry-sample Borings. Instead of relying for information on the material carried out by the wash water, a better method is to obtain samples by driving a pipe or spoon into the soil at the bottom of the hole. This is known as dry-sample boring, not that the sample obtained is necessarily dry, but to differentiate this method from that in which samples are obtained from the wash water. The boring work is carried on in the same manner as for wash borings. When it appears that a change in the character of the soil is taking place, the wash pipe is raised a short distance and pumping continued until the wash water comes out clear.

The string of wash pipe is then removed from the casing and a piece of open 1-in. pipe, or some form of spoon, is substituted for the chopping bit. The wash pipe is then returned to the hole and driven for a sample, after which the pipe is lifted and the sample taken out and placed in a bottle. The bottle is then marked with the boring number and the depth from which the sample was taken. Wash boring is then continued until another change in material is encountered and the process is then repeated.

Samples taken this way cannot be considered as undisturbed samples, but they do give the composition of the soil. The sample may not give a true picture of the degree of consolidation, but a rough approximation of this may be arrived at by noting the resistance to penetration of the wash pipe in each stratum, measured either in terms of the static load necessary to force the pipe into the stratum or in terms of the foot-pounds of energy involved.

1-8. Test Pits. There is no method of soil exploration that gives so complete an answer as a test pit. Although usually more expensive than other methods, especially where considerable depths are involved, it offers the great advantage of making it possible to examine not only the variation in character of the earth encountered but also the degree of consolidation in its natural state. Another advantage is the convenience afforded in obtaining undisturbed samples for laboratory tests.

The pit is usually sunk by hand excavation and is ordinarily square in section and large enough for comfortable working conditions inside. A pit can be dug about 9 ft. deep with no tools, other than picks and shovels. For deeper pits a tripod or A-frame, with rope and pulley or a power hoist, is required for lifting the excavated material. For dry pits in caving soil and in wet pits, sheathing is necessary. Where the soil is dry and reasonably firm, horizontal sheeting may be used, but it is better practice to use vertical sheet piling driven in advance of excavation.

Figure 1-8*a* illustrates a type of test pit used by the New Mexico State Highway Department. The material required for a depth of 14 ft. is about 1,500 ft. b.m., the pit being 4 by 6 ft. in plan. Test pits will usually cost about \$10 per foot of depth. For example, assuming that the labor cost of excavating will be \$2 per cubic yard, that there will be 1 cu. yd. per ft. of depth, and that the sheeting will cost \$75 per M ft. b.m. in place, with 100 ft. b.m. per ft. of depth, the total cost will be \$9.50 per foot of depth, exclusive of

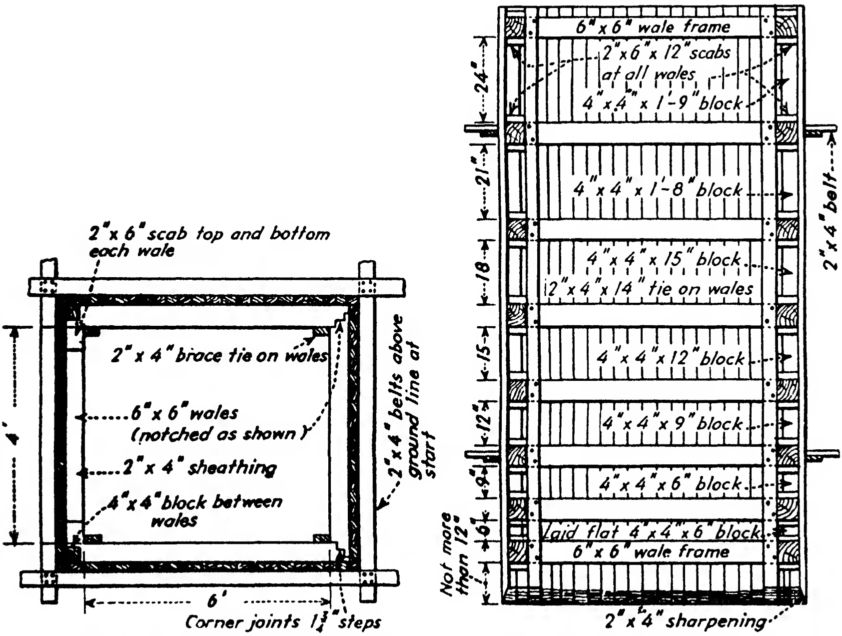


FIG. 1-8*a*.—Test Pit Used by the New Mexico State Highway Department.

such items as interest, overhead, and depreciation of tools and equipment.

Deep test wells are sometimes drilled by power, a mobile power auger (Art. 12-6) being used. The hole is made circular in section and need not be more than 2 or 3 ft. in diameter, or just enough to permit carrying on inspection and sampling operations. In California unlined wells as deep as 70 ft. have been successfully used.[1]

1-9. Undisturbed Sampling. In order to make laboratory tests of the physical characteristics of soils (see Chap. II), it is necessary to secure samples that have been subjected to a minimum of dis-

[1] *Eng. News-Record*, vol. 117, p. 194, Aug. 6, 1936.

turbance. The disturbance to which a soil may be subjected during sampling may be classified as follows:[1]

a. Changes in stress conditions
b. Changes in water content or void ratio
c. Disturbance of the soil structure
d. Changes in thickness of the soil layers
e. Mixing of the soil layers

The most satisfactory results are obtained where the use of a mechanical device is not required, as in the case of samples taken at the surface of the ground or in test pits. In this case, if the soil possesses some cohesion, a trench may be excavated to form a ledge or bench from which to cut the sample. The sample is first roughed out and then trimmed with a wire saw or with a hack saw. A wooden or metal box, without top or bottom, is lowered over the sample with some clearance all around. The clearance space is filled with damp sand carefully tamped in place. The top of the sample is then trimmed to about 1 in. below the top of the box, after which the space is filled with damp sand and the cover placed. The soil column is then cut off below the bottom of the box with a square spade or metal plate, after which the box is turned over, the excess soil removed, the sand layer placed, and the bottom cover fastened on. For firm cohesive soils the clearances can be reduced and paraffin used in place of damp sand.

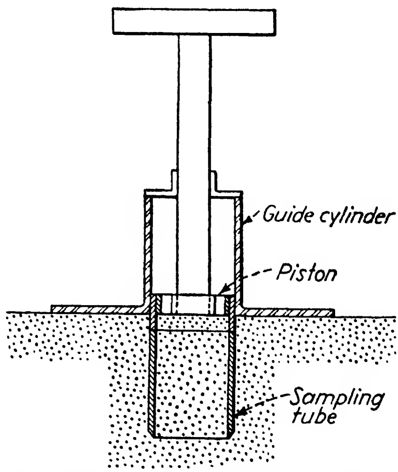


FIG. 1-9*a*.—A Sampler Used for Cohesionless Soils.

Where the soil has only slight cohesion, it is better to work the box down over the column by excavating and trimming only a little in advance of the box. In this case the sample fills the entire box, and it is better to use a cylindrical container since in a square box it is difficult to avoid disturbing the corners.

[1] See report entitled The Present Status of the Art of Obtaining Undisturbed Samples of Soils, March, 1940, by M. Juul Hvorslev. This is a report of the Committee on Sampling and Testing, Soils Mechanics and Foundations Division, American Society of Civil Engineers.

In entirely cohesionless soils or in other soils where rapid sampling is desired, surface sampling may be done by forcing short tubes into the soil. The sampler usually consists of a beveled piece of steel tubing. The details of the apparatus differ considerably, Fig. 1-9*a* representing a type widely used in Germany. The sampling tube and the piston used to force it into the soil are guided by another tube which is welded to a foot plate. The sampler is taken from the ground by removing the soil around it, the top and bottom of the tube being sealed with celluloid caps and adhesive tape.

Samplers used below the surface of the ground may be divided into two classes, (*a*) those used without cased bore holes and (*b*) those used with cased bore holes. Where cased bore holes are not used, the entrance of soil into the sampling tube during its driving to sampling depth may be prevented by temporary closure by means of a piston. This type is illustrated in the sampler designed by O. J. Porter and extensively used by the California Division of Highways.

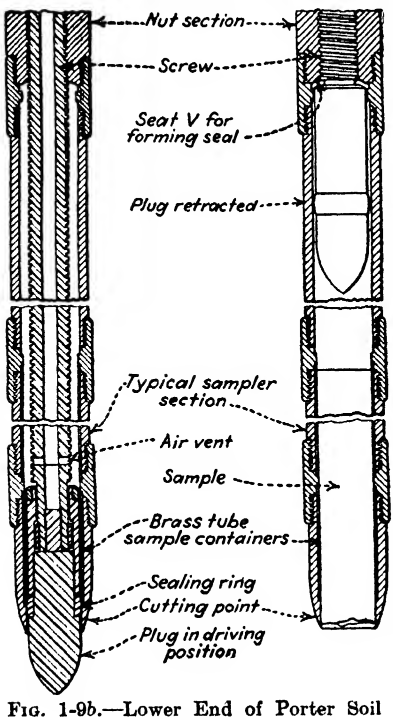


FIG. 1-9*b*.—Lower End of Porter Soil Sampler.

The lower end of the sampling tube is shown in Fig. 1-9*b*. The section on the left shows the bullet-shaped plug in the cutting edge. This is the position for driving. After being driven to the desired depth, this plug is withdrawn from the cutting edge by turning the screw, which is actuated by an inside rod from the surface. With the plug in the retracted position the sampler is forced down still farther, thereby filling the brass tube sample containers. While this farther advance of the sampler is in progress, air and water can escape around the retracted plug. However, after the samples have been forced into the tubes within the tip, the plug is withdrawn still farther to effect a seal by closing tightly against the seat V. This protects the space in which the samples are lodged against suction that would tend to withdraw them while the device is being pulled up to the surface. As soon as the sampler is out of the hole, the tip and the

12-inch sampler sections are unscrewed, the several sample containers, each 2 inches long, are removed, weighed, capped, and sealed for shipment to the laboratory.[1]

In Fig. 1-9*c* there is shown a general arrangement of the sampler assembly. Casing is necessary only where free-flowing sand is encountered or where the skin friction is too great to permit easy driving and withdrawal of the sample. Practically continuous 2-in. core samples have been taken by this type of sampler from the surface of the ground to depths of 100 to 220 ft. at a total cost of from \$1 to \$2.50 per foot, as compared with costs of from \$5 to \$8 per foot for the larger samples described in the following paragraphs.

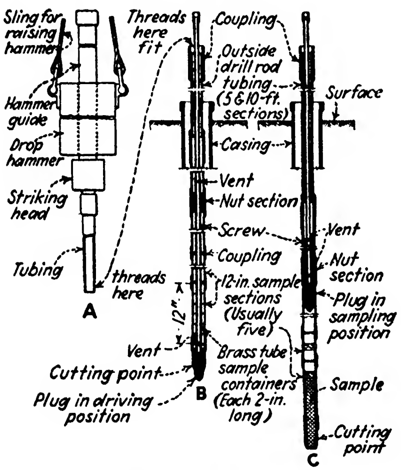


FIG. 1-9*c*.—Complete Assembly of Porter Soil Sampler.

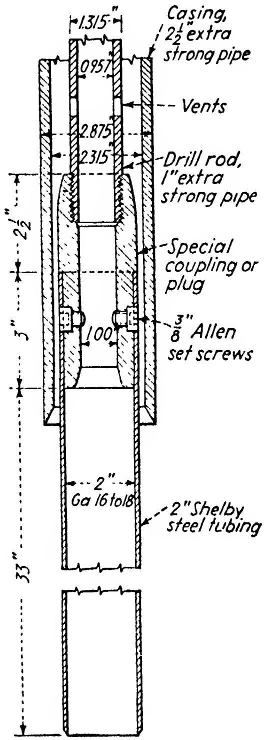


FIG. 1-9*d*.—Shelby Tubing Sampler by H. A. Mohr.

Drilling and casing of a bore hole, although more expensive, is most commonly used to reach soil layers at considerable depths below the ground surface. It results in the least disturbance to the soil and should always be used where large undisturbed samples are required. One of the simplest types of samplers, designed by H. A. Mohr,[2] is shown in Fig. 1-9*d*. A special coupling or plug fits snugly in the tubing and is provided with a collar through which the driving force is transmitted to the tubing. The force for withdrawal of

[1] *Eng. News-Record*, vol. 116, p. 804, July 4, 1936.
[2] Taken from bulletin noted in first reference of this article.

the tube is transmitted through two Allen setscrews. This type of sampler is satisfactory for use in cohesive soils for preliminary undisturbed sampling, but, for final undisturbed sampling where a minimum of disturbance is necessary, larger samplers of a minimum diameter of 4 in. and equipped with special devices must be used.

Of the four forms of disturbances to which soil may be subjected in sampling, the first one—a change in the stress conditions—cannot be avoided. A small cubic element of saturated air-free soil in the undisturbed ground is under compressive forces on all faces due to the pressure from the surrounding particles of soil, as well as to the hydrostatic pressure in the pore water. When this soil sample is

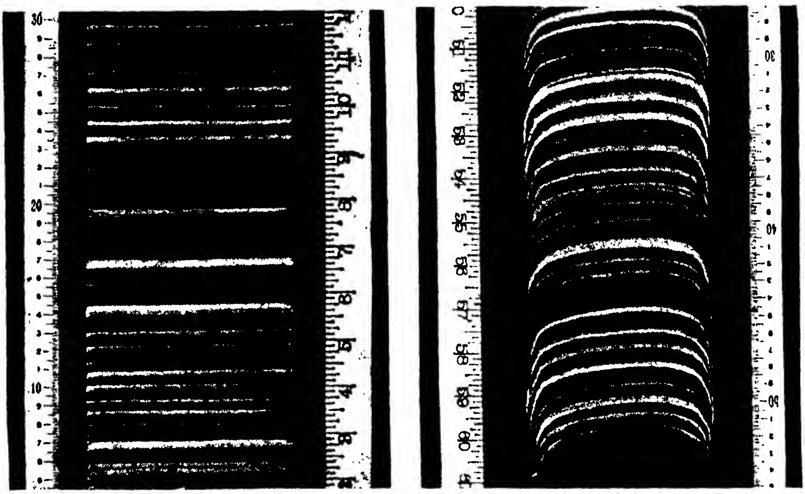


FIG. 1-9*e*.—Vertical Sections of Samples with Undistorted and Distorted Soil Layers. (*Courtesy of M. Juul Hvorslev.*)

removed from its position in the ground and exposed to atmospheric pressure only, it increases slightly in volume due mainly to the elastic expansion of the pore water; but further expansion cannot take place without an increase of water content or admission of air to the pores. Hence the sample is left in a state of internal stress, water menisci forming on the surface with a curvature such that the capillary forces created will be large enough to balance the internal elastic forces, provided the adhesion between the menisci and the walls of the pores is not exceeded.

Changes in water content, particularly as between different parts of a sample, may easily occur. In the sampling operation the upper part of the sample may be subject to swelling, while the lower part is compressed. Hence there will be a tendency for the water to move upward in the sample. Any disturbance in the soil struc-

ture will tend to cause a migration of the water. After the specimen has been removed from the sampling tube, the water content may be decreased through evaporation.

In sampling, the soil structure may be gravely disturbed and distorted or only slightly so, depending on (*a*) the care taken, (*b*) the type of sampler used, and (*c*) the type of soil. The degree of disturbance may vary over the cross section and over the length of the

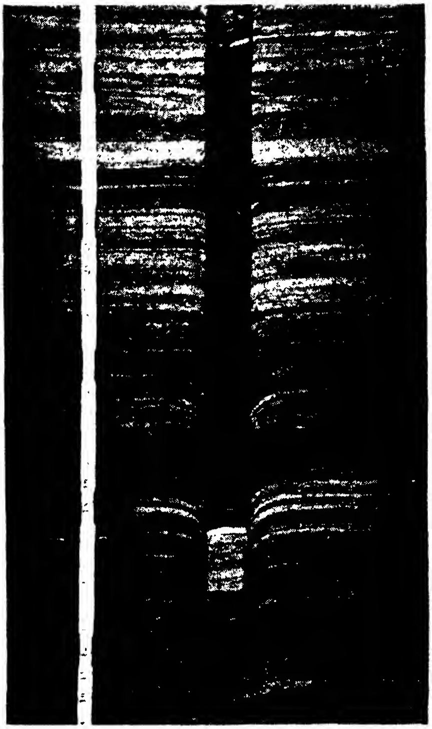


FIG. 1-9*f*.—Vertical Section Showing Soil Distortion near Bottom. (*Courtesy of M. Juul Hvorslev.*)

sample. In careful sampling it should be confined to a thin layer on the surface of the specimen. In Fig. 1-9*e* there are shown vertical sections of two samples, the one on the left being undistorted and that on the right badly distorted.

In plastic soils many sampling methods result in an appreciable change in the thickness of the soil layers. In forcing the sampler into the ground, at the start the volume of earth displaced by the sampler shell may enter the sampler to increase the height of the soil cylinder over the depth penetrated by a percentage depending on the ratio of the area of the cross section of the shell to the cross

section of the specimen. With further penetration the increase in friction of the specimen along the inside wall tends to compress the soil below the edge of the sampling tube (see Fig. 1-9*f*) as a result of which less than the true length of the sample enters the sampler.

Mixing of soil layers and separation of fine and coarse material sometimes result from carelessness in advancing and cleaning the bore hole, especially where jetting is used. Manifestly, laboratory tests made on such a specimen are worthless in determining the general characteristics of the soil profile.

M. Juul Hvorslev, in a paper entitled Progress in the Development of Improved Methods for Obtaining Undisturbed Samples of Soil, read before the American Society of Civil Engineers, lists the following as desirable characteristics in a sampler:

1. The area ratio of the sampler—ratio of area of cross section of sampler wall to area of cross section of sampler—should be reduced to a minimum. Where this ratio exceeds 10 to 20 per cent, excess entrance of soil is likely to occur.

2. The cutting edge should provide an inside clearance of 1 to 2 per cent to reduce inside friction, in order to eliminate the downward deflection of the soil layers below the cutting edge. When the clearance exceeds $2\frac{1}{2}$ per cent, excessive lateral expansion of the lower part of the sample may result, or it may be difficult to retain the sample in the tube during withdrawal.

3. The inside of the sampling tube and shoe should be smooth and continuous, with no protruding edges, and with grooves avoided as far as possible, in order to reduce friction and distortion of soil layers.

4. The outside of the sampler shoe should be smooth and rounded and with a small taper toward the cutting edge, in order to reduce the magnitude and concentration of the point resistance and thereby reduce the tendency toward entrance of excess soil into the sampler. The cutting edge should be sharp.

5. The best samples are obtained when a fast continuous drive—0.5 to 1 ft. per sec.—is used to sink the sampler. This can be done by the use of a hydraulic ram having a stroke at least equal to the desired depth of penetration, or by the use of block and tackle. A single heavy blow gives good results, as does shooting, but slow ramming or driving with intermittent blows is not so satisfactory.

6. The length of a sample taken in a single drive should not exceed fifteen times the diameter of the sampler and should equal this value only when a properly designed sampler is used under the best methods of driving.

Many forms of samplers are in use at the present time, but they are all more or less in the experimental stage. Figure 1-9*g* illustrates a design developed by Casagrande, Mohr, and Rutledge,

which provides not only for getting undisturbed samples into the barrel but also for holding the same in during the raising of the apparatus. The sample is cut free by means of piano wire which fits in a groove in the shoe. The top of the sampler is tapped for two air-hose connections, the one on the left to provide a vacuum above the sample during its withdrawal and the one on the right to admit compressed air on the underside of the sample. Another method of holding the specimen in the sampler is to use flap valves or other mechanical devices at the bottom.

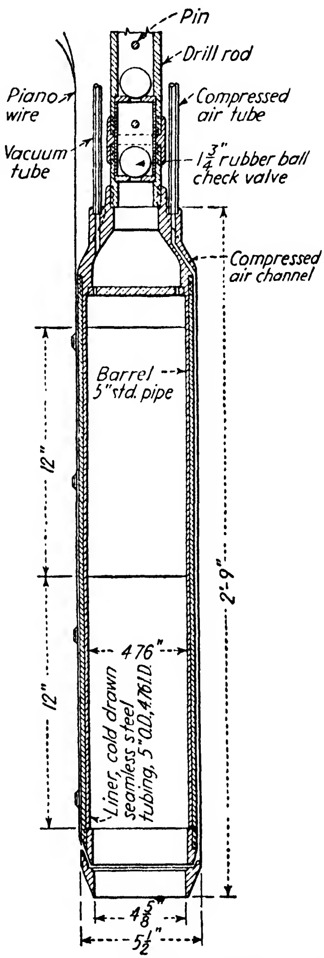


FIG. 1-9g.—Soil Sampler Developed by Casagrande, Mohr, and Rutledge.

In coarse cohesionless soils it is very difficult to force the sampler into the soil by means of a steady pressure. In this case it may be better to use the core-boring method in place of drive sampling. In core boring a double-tube core barrel is used, in which the inner or stationary barrel extends a short distance below the outer or rotating barrel.

Figure 1-9*h* illustrates the core-boring sampler[1] used on investigational work on the Denison Dam in Texas. The hydraulic feed of the drill rig acts as a jack to force the sampler down, while the rotating outer barrel reams out the hole around the sampling shoe. The circulating drilling mud serves as a jet to remove surplus material from the hole. One section of casing was used at the top, but, below this, drilling mud was used in place of casing, the hole being kept full of mud at all times to support the walls below the casing. The best drilling mud to use is that which is thixotropic, that is, one that acquires a temporary set when undisturbed but becomes fluid when agitated.

[1] From article entitled Improved Sampler and Sampling Technique for Cohesionless Materials, by H. L. Johnson, *Civil Eng.*, vol. 10, p. 346, June, 1940.

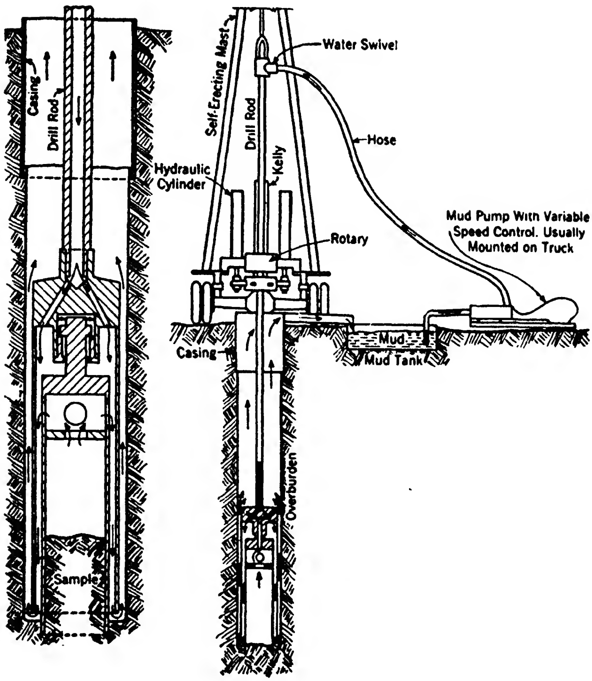


FIG. 1-9h.—Rotary Drill Type of Sampler.

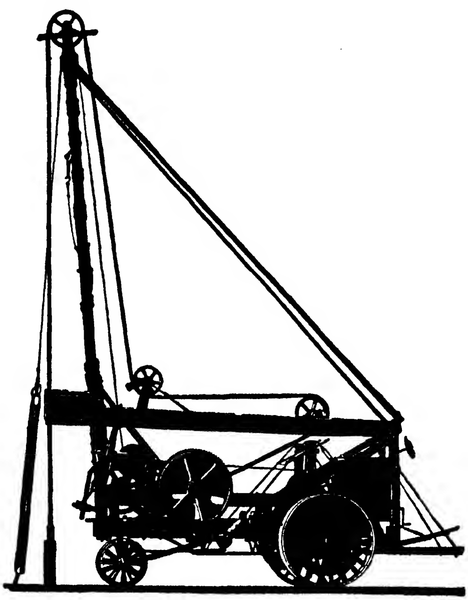


FIG. 1-10a.—Churn or Percussion Drilling Outfit. (*Courtesy of Keystone Driller Co.*)

FIG. 1-10*b*. Percussion Core Driller. (*Courtesy of Keystone Driller Co.*)

1-10. Churn or Percussion Drilling. In soil exploration work, when rock is reached, it is desirable to drill some distance into the same in order to eliminate any possibility of mistaking a boulder or thin layer of rock for bedrock. Drilling through rock is usually done by one of the following three methods: (*a*) churn or percussion drilling, (*b*) diamond drilling, and (*c*) shot or tooth-cutting drilling.

The ordinary churn or percussion drill (Fig. 1-10*a*), widely used for drilling water and oil wells, is particularly valuable in exploration work through boulder-strewn gravel. It will drill through rock and, of course, through soil, but only in the softer rocks can cores be obtained. The machine consists of an apparatus for raising and lowering, with a churning motion, a string of tools consisting of a hardened steel chopping bit screwed into a stem, which in turn is attached to a set of jars held by a rope (Fig. 7-9*c*). These jars are for jerking the bit loose when it sticks in the bottom of the hole.

In using the churn-drill method it is generally necessary to case that part of the hole above bedrock. The tools, churning up and down, form a slurry of the displaced soil or broken rock mass and water, the water being poured into the hole as sinking progresses. From time to time the hole is bailed out by means of a sand pump (Fig. 7-9*d*), which consists of a section of iron or steel pipe with a valve near the bottom.

When a rock stratum is reached at which a core is desired, a special core drill replaces the steel bit and stem. The bit of this core drill is similar to the Davis cutter (Fig. 1-12*d*) and inside the drill rod there is a core barrel (Fig. 1-10*b*). This barrel is free to move up and down inside the drill rod and may extend down 4 ft. below the bottom of the bit. As the drilling stroke is only about 18 in., the barrel is not lifted off the bottom by the motion of the drill. After drilling about 2 ft., the tools are withdrawn and along with them the core, which usually consists of a number of irregularly shaped fragments. The churn drill is not suitable for drilling in hard rock but is very satisfactory in material

having the hardness of bituminous coal or fire clay. The cost will generally range between $5 and $15 per foot.

1-11. Core Drilling with Diamonds. In core drilling with diamonds a rotary drill outfit is used. The drill itself consists of a hollow bit (Fig. 1-11*a*), in which two rows of black diamonds are set around the edges, in such a manner that all the cutting is done by them, and with a small clearance both inside and outside. The metal of the bit is rather soft to enable placing the diamonds in position. As the drill spindle is rotated, water is forced down through the hollow drill rod and bit to keep the latter cool, and in

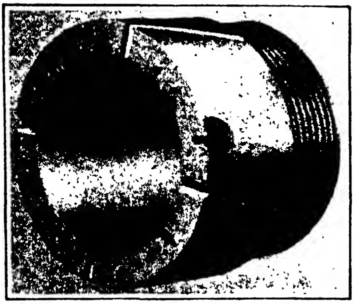


FIG. 1-11*a*.—Diamond Bit. (*Courtesy of Sullivan Machinery Co.*)

FIG. 1-11*b*.—Core Shell, Bit, and Lifter of Diamond Drill. (*Courtesy of Sullivan Machinery Co.*)

passing upward outside of them the water carries the cuttings to the surface.

The bit is screwed to the core barrel and that in turn to the drill rod. The bit cuts an annular channel, and the core formed within is protected by the descending core barrel. At intervals the core is broken off by a special device and lifted out along with the barrel. Figure 1-11*b* shows an assembly consisting of core barrel, bit, and lifter. Two kinds of black diamonds are used for this work, carbon being set for cutting hard rock and bortz for soft rock. The bortz is as hard as the carbon but not so tough. For medium rock, half carbon and half bortz may be used. The diameter of the core varies from 1 to 2 in. for foundation explorations, larger diameters being customary for wells, tunnels, and deep-mine prospecting.

Where surface material overlies the rock, a casing is first driven to rock, as shown in Fig. 1-11*c*, and the soil inside the casing removed, usually by the wash-boring method. The diamond drill may then be set up over the casing, and drilling operations are

begun. Figure 1-11*d* illustrates a Sullivan No. 7 core drill with screw-feed swivel head and integral circulating pump drilling at an angle. The swivel head is mounted in the clear so that it can be swung for any angle of drilling. While rods are being pulled, the swivel head is unlatched and swung aside, which automatically

Fig. 1-11*c*.—Core Drilling Outfit Mounted on a Truck. (*Courtesy of Sullivan Machinery Co.*)

disconnects the drill spindle from the drive shaft. The power unit shown in the illustration is a gasoline engine. Electric motors and compressed-air motors are also used.

The rate of sinking and cost of diamond-drill work depend on the character of the rock and the amount of overburden. The following data relate to drilling holes for the Deep Waterways Survey where 25 holes were sunk to an average depth of 98.5 ft. The rates for

drilling in different kinds of rock, in feet per hour, were as follows: quartzite, 1.7; limestone, 2.5; sandstone, 3; and shale, 5. The total length of casing sunk was 552 ft., requiring 325 hr., hence the rate was 1.7 ft. per hr.; whereas the total length of holes drilled in rock was 1,910 ft., requiring 753 hr., which makes the average rate in rock 2.54 ft. per hr. The total carbon loss was $25\frac{6}{64}$ carats in drilling

Fig. 1-11*d*.—Core Drilling at an Angle. (*Courtesy of Sullivan Machinery Co.*)

1,688 ft. of rock, making the cost for diamonds 47.7 cts. per foot, at $36.50 per carat. The total cost of drilling averaged $3.18 per foot.

In drilling 82 holes, totaling 1,639 ft., for 12 highway bridge foundations in West Virginia in which the drilling was carried from 3 to 5 ft. into bedrock, the cost per foot varied from $1.20 to $5.02, with an average of $2.39. The breakdown average costs per foot were as follows: labor, fuel, and miscellaneous, $1.72; renewal of equipment, 17 cts.; moving, 18 cts.; and depreciation, 32 cts. The

figure for depreciation is too high, as it was based on a life of only 3 years for the equipment, whereas the actual life was at least double this.

1-12. Core Drilling with Shot and Tooth Cutting. The greatly increased demand for black diamonds for drilling in mine prospecting and for other purposes led to such a rise in price that other methods of core drilling have been developed. In one type of drill, chilled-steel shot, usually irregular in shape, are made to travel under the edge of a hollow soft-steel bit which rotates and exerts a vertical pressure on the shot, thereby causing them to mill away the rock. As in diamond drilling, the bit is screwed into the core barrel, which in turn is screwed into the hollow drill rod. One side of the bit has a diagonal slot in it (Fig. 1-12*a*) to aid the shot to work freely under the bit and to permit some of the water from the jet to escape without passing under the edge of the bit.

FIG. 1-12*a*. Soft-steel Bit Used in Shot Drilling. (*Courtesy of Ingersoll-Rand Co.*)

The shot varies in size from dust to particles as large as duck shot. This shot is automatically fed to the drill at a uniform rate during drilling.

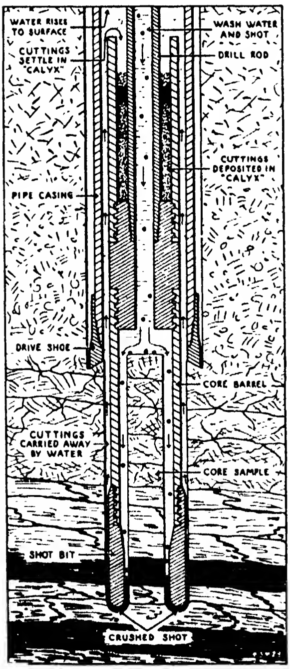


FIG. 1-12*b*.—Shot Drill Showing Calyx or Sludge Receiver. (*Courtesy of Ingersoll-Rand Co.*)

A special feature of one make of shot drill (Fig. 1-12*b*) is the calyx or sludge receiver. It is formed by a tube surrounding the drill rod above the core barrel, in which are deposited the chips or sludge, on account of the sudden decrease in velocity of the upward current of water. The cuttings thus received form a duplicate record of the strata penetrated. If sufficient water is used to bring the cuttings to the surface, its velocity is so great as to wash the shot away from under the bit.

To remove the core, small gravel is dropped into the hollow drill rod, where it wedges between the core and the inner wall of the bit near the bottom when the drill is given a few turns. This breaks the core and permits lifting it.

Cores cut by shot drills are usually larger than those cut by diamond drills, ranging from $1\frac{1}{2}$ to 20 in. and larger in diameter. The largest cores are required for purposes other than bridge-foundation explorations. Cores having diameters of 4 to 5 in. can be extracted practically as cheaply as the smaller ones, while the rate of progress is equally as fast. Except for the harder rocks, the shot drill is often more economical than the diamond drill.

The successful operation of the shot drill requires careful regulation of the water flow in order that the cuttings may be removed without displacing the shot. To obtain effective cutting, careful attention must also be given to the vertical pressure applied. The most serious difficulty in shot drilling is due to crevices in the rock through which the shot may be lost. Where these are present, some means must be found of artificially sealing the openings, or perhaps a toothed bit may be substituted to drill past the crevice.

FIG. 1-12*c*.—Calyx Cutter. (*Courtesy of Ingersoll-Rand Co.*)

FIG. 1-12*d*.—Davis Cutter. (*Courtesy of Ingersoll-Rand Co.*)

In another type of rotary drill, steel bits are used with different forms of teeth. Figure 1-12*c* illustrates the Calyx cutter, which has removable steel teeth. It is used in soft and moderately hard rocks where the shot drill would not be suitable. Another tooth-drill cutter, known as the "Davis cutter," illustrated in Fig. 1-12*d*, is used principally in drilling through soil overlying rock.

1-13. Exploration Reports. The results of exploration borings should be presented in a report that is complete, accurate, and concise. These results can best be accomplished by furnishing the drilling foreman with a form having blank spaces for all desired information. In general the following information should be obtained:

a. Locate position of bore holes on a survey plat of the ground, numbering holes in proper order.

b. Give starting grade of each boring, this grade to be referenced to a definitely established base.

c. Show cross sections of the soil strata, noting elevation of ground surface and the depth below starting grade of the top of each stratum, identifying the several strata as nearly as possible as to composition.

d. Note elevation of ground-water surface.

e. Indicate degree of compactness of the soil, preferably by the resistance to penetration of a sampling pipe.

f. Note presence of boulders and obstructions of any type.

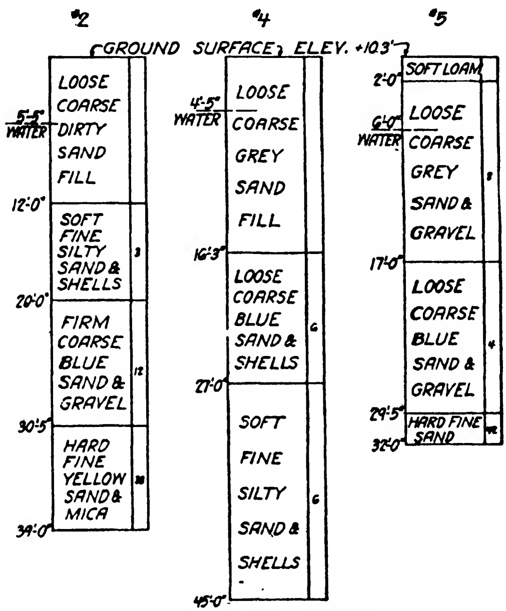


FIG. 1-13*a*.—Log of Borings.

Figure 1-13*a* illustrates a properly prepared log of three borings.[1] The figures in the right-hand column indicate the number of blows required to drive a sampling pipe 1 ft., using a 140-lb. weight falling 30 in.

1-14. Determination of Bearing Capacity. On completion of the exploration survey the engineer will be in possession of the following information: (*a*) elevation of bedrock (if same is reached); (*b*) character of soil (hardpan, gravel, sand, clay, silt, or combination of these); (*c*) presence or absence of boulders, sunken logs, or other obstructions; and (*d*) elevation of ground-water surface.

The foundation will usually be carried to bedrock where the same is found at a reasonable depth. As to what is a reasonable

[1] Graduate School of Engineering, Harvard University, *Bull.* 208.

depth will depend on the character and size of the structure, as well as on difficulties involved in excavation operations. For deep foundations, caissons are usually used; or, if the structure is on land and ground-water conditions are favorable, the open-well method may be employed.

Where rock is not available, there must be determined (*a*) the best type of foundation and (*b*) the proper dimensions of same. The types commonly used are enumerated in Art. 1-1 and discussed in the following chapters. Having selected the type, the next step is to determine the dimensions to the end that both safety and economy will be served. The plan area of the foundation will depend largely on the unit soil bearing pressure adopted. The unit must be so chosen as to keep settlement down to a safe limit.

A large fund of practical information on safe unit bearing values has been collected by the engineering profession through observations on the behavior of existing structures and also by the making of load-settlement tests as a preliminary to construction work. This information is available in the form of tables of permissible unit bearing loads for different types of soils, which tables appear in most standard bridge and building specifications, as well as in building codes.

Within the last few years a small group of investigators headed by Charles Terzaghi has opened up the field of soil mechanics, which has enabled the engineer to put foundation design on a more rational basis. A brief outline of some of the fundamentals of soil mechanics as applied to the field of structural engineering is given in the following chapter.

Any table of permissible unit loads must be used with caution, for the bearing capacity of a soil depends on many factors in addition to the soil type, such as (*a*) degree of compaction, (*b*) water content, (*c*) size of loaded area, and (*d*) depth below ground surface. The soil may be a mixture of two or more classes, or it may be stratified and contain a number of layers of varying types.

Loading tests may give a great deal of helpful information. These may be supplemented by laboratory tests on small samples. However, laboratory tests based on our present knowledge of soil mechanics must be confined to situations where we have thick beds of uniform materials, such as clay or sand. The difficulty of applying and interpreting laboratory tests becomes very great when the site is underlaid by irregular lenses of varying and diverse soils, particularly when these lenses are largely composed of mixtures of sand and clay, whose mechanics are not clearly understood. Even

though the laboratory quantities may be determined with precision, yet, because of the variation and discontinuity in strata, any interpretation may be subject to grave errors.

1-15. Values of Bearing Capacity. The Boston Building Code (1937) contains one of the latest tables of permissible unit pressures. Although some of the soil types are more or less local in character, most of the values given are of general application. These permissible unit pressures are to be used in connection with dead and live load; where wind stresses are included, the units may be increased 25 per cent.

Class	Material	Value, allowable bearing, tons per square foot
1	Massive bedrock without laminations, such as granite, diorite, and other granitic rocks, and also gneiss, traprock, felsite, and thoroughly cemented conglomerates, such as Roxbury pudding stone, all in sound condition (sound condition allows some cracks)	100
2	Laminated rocks such as slate and schist in sound condition (some cracks allowed)	35
3	Shale in sound condition (some cracks allowed)	10
4	Residual deposits of shattered or broken bedrock of any kind except shale	10
5	Hardpan	10
6	Gravel, sand-gravel mixtures, compact	5
7	Gravel, sand-gravel mixtures, loose	4
8	Sand, coarse, loose; sand, fine, compact	3
9	Sand, fine, loose	1
10	Hard clay	6
11	Medium clay	4
12	Soft clay	1
13	Rock flour, shattered shale, or any deposit of unusual character not provided for herein	Value to be fixed by the commissioner

These figures shall govern except where load tests are made (Art. 1-16), and they are subject to the following limitations:

a. The tabulated bearing values for rocks of classes 1 to 3, inclusive, shall apply when the loaded area is less than 2 ft. below the lowest adjacent surface of sound rock. Where the loaded area is more than 2 ft. below such surface, these values may be increased 20 per cent for each foot of additional depth but shall not exceed twice the tabulated values.

b. The allowable bearing values of materials of classes 4 to 9, inclusive, may exceed the tabulated values by $2\frac{1}{2}$ per cent for each foot of depth of the loaded area below the lowest ground surface immediately adjacent, but they shall not exceed twice the tabulated values. For areas of foundations smaller than 3 ft. in least lateral dimension, the allowable bearing values shall be one-third of the allowable bearing values multiplied by the least lateral dimension in feet.

c. The tabulated bearing values for classes 10 to 12, inclusive, apply only to pressures under individual footings, walls, and piers. When structures are founded in, or are underlain by, deposits of these classes, the total load over the area of any one bay or other major portion of the structure, minus the weight of the excavated material, divided by the area, shall not exceed one-half the tabulated values.

d. Where the bearing materials directly under a foundation overlie a stratum having smaller allowable bearing values, these smaller values shall not be exceeded at the level of such stratum. Computation of the vertical pressure in the bearing materials at any depth below a foundation shall be made on the assumption that the load is spread uniformly at an angle of 60 degrees with the horizontal; but the area considered as supporting the load shall not extend beyond the intersection of 60-degree planes of adjacent foundations.

This code defines bearing materials as follows:

a. Rocks:

Shale—a laminated, fine-textured, soft rock composed of consolidated clay or silt, which cannot be molded without the addition of water, but which can be reduced to a plastic condition by moderate grinding and mixing with water.

Slate—a dense, very fine-textured, soft rock which is readily split along cleavage planes into thin sheets and which cannot be reduced to a plastic condition by moderate grinding and mixing with water.

Schist—a fine-textured, laminated rock with more or less wavy cleavage, containing mica or other flaky minerals.

b. Granular Soil:

Gravel—an uncemented mixture of mineral grains $\frac{1}{4}$ in. or more in diameter.

Sand—a type of soil possessing practically no cohesion when dry, and consisting of mineral grains smaller than one-quarter inch in diameter.

Coarse sand—a sand consisting chiefly of grains which will be retained on a 65-mesh sieve.

Fine sand—a sand consisting chiefly of grains which will pass a 65-mesh sieve.

Compact gravel, compact sand—deposits requiring picking for removal and offering high resistance to penetration by excavating tools.

Loose gravel, loose sand—deposits readily removable by shoveling only.

c. Cohesive Soil:

Hardpan—a thoroughly compact mixture of clay, sand, gravel, and boulders, for example, boulder clay; or a cemented mixture of sand or of sand and gravel, with or without boulders, and difficult to remove by picking.

Clay—a fine-grained, inorganic soil possessing sufficient cohesion when dry to form hard lumps which cannot readily be pulverized by the fingers.

Hard clay—a clay requiring picking for removal, a fresh sample of which cannot be molded in the fingers, or can be molded only with the greatest difficulty.

Medium clay—a clay which can be removed by spading, a fresh sample of which can be molded by a substantial pressure of the fingers.

Soft clay—a clay which, when freshly sampled, can be molded under relatively slight pressure of the fingers.

1-16. Load Tests. The equipment for a loading test is simple, usually consisting of a bearing plate resting on the undisturbed soil, the load being transmitted to the plate from a loaded platform through a vertical strut. The load may consist of any available heavy material, such as pig iron, iron castings, steel rails, bags of sand or cement, etc.

Figure 1-16*a* illustrates a typical test outfit for hardpan, on which the platform is held in position by wedges, loosely placed against the sheeting. The settlement is measured by taking readings on a steel tape at the top of the shaft. When the platform is placed above the natural ground surface, it may be braced conveniently by extending the post above the height to be occupied by the loading and into a loosely fitting collar which is held in position by four inclined shores; or four vertical timbers supported by shores may be placed so as to correspond to the four sheeting planks which take bearing from the wedges in Fig. 1-16*a*.

Figure 1-16*b* shows the test results made on stiff blue-clay hardpan 20 ft. below the surface at Lansing, Mich. Apparatus for the test consisted of a 12- by 12-in. post placed upon a steel plate on the hardpan and carrying a 60-ton hydraulic jack, upon the head of which rested the bottom beam of a wooden bin which was held in position by vertical guides.

Loading was increased in 2-ton increments at 15-min. intervals up to a total load of 92,000 lb. per sq. ft., deflection readings being taken before and after each increase. Circular bearing plates were used under the

compression post, with areas of 1, 2, and 4 sq. ft. for the three tests, respectively. Settlement was measured by a lever-arm device which multiplied actual settlements by five and supplied an accuracy of 0.01 in. The

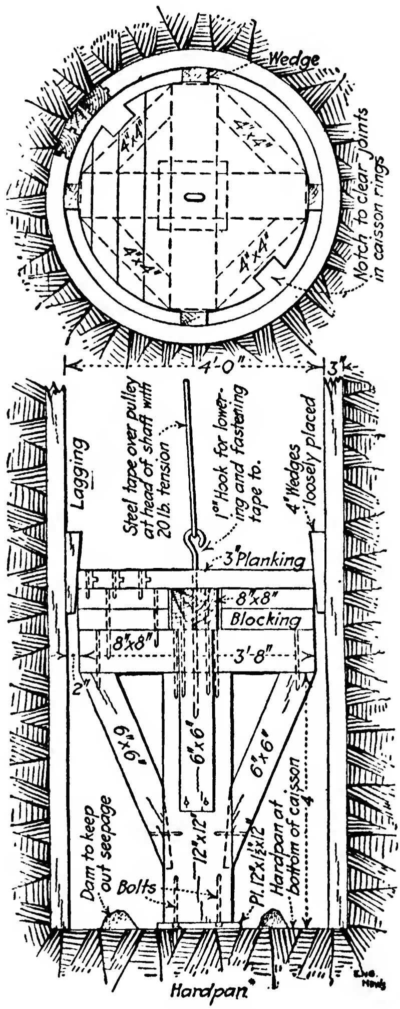


FIG. 1-16a.—Load-testing Outfit.

circular areas were tested under conditions of total confinement due to overburden, the plates being set at the bottom of 3-ft. holes having the same diameters as the plates.[1]

As an illustration of soil testing under difficult conditions the work on the Suisun Bay (Calif.) bridge may be cited. The depth

[1] *Eng. News-Record,* vol. 108, p. 330, Mar. 3, 1932.

of water varied from 25 to 55 ft., with rock from 115 to 141 ft. below mean low tide, mud, sand, and clay overlying the rock. Drilling platforms supported on piles were erected and steel cages made of four angle irons laced on four sides were driven into the bottom. Guided within the cages, 10-in. well casings were put down, all drilling being done inside these casings. Bearing tests were made by using extra-heavy 5-in. pipe, to the lower end of which a 6- by 6-in. hardwood block was fastened. The pipe was lowered into the well until the block rested on the material to be tested and the top of the pipe was approximately 3 ft. above the

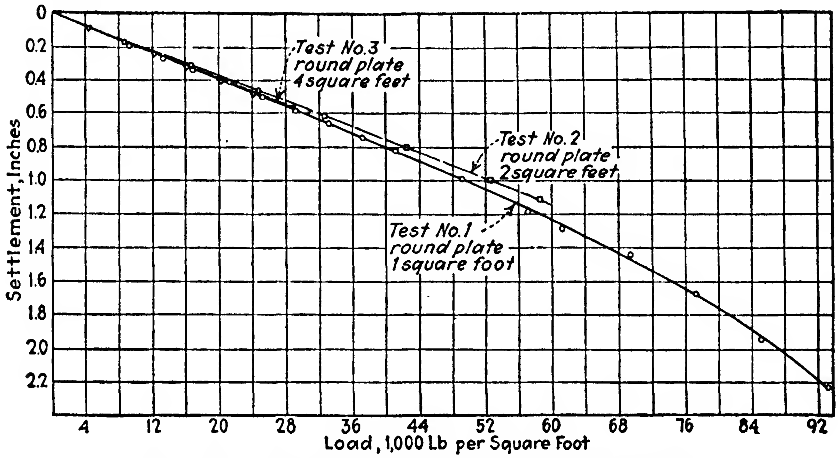


Fig. 1-16*b*.—Load Settlement Curves for Tests on Hardpan.

platform floor. Care was taken to have the bottom of the hole at least 2 ft. below the base of the 10-in. casing. The pipe was then loaded by putting water in a barrel supported on a lever, the lever-arm ratio being 1:15.

The Boston Building Code contains the following specifications for making load tests (see also Art. 2-16).

a. For bearing materials of classes 1 to 5, inclusive (Art. 1-15), the loaded area shall be at least 1 sq. ft. and for other classes at least 4 sq. ft. For materials of classes 6 to 13, inclusive, the loaded area shall be the full size of the pit and at such depths that the ratio of the width of the loaded area to its depth below the immediately adjacent ground surface is the same as the larger of the following two values:

(1) Ratio of the width of any footing to its depth below the immediately adjacent ground surface.

(2) Ratio of the width of the entire foundation or group of footings to its depth below the average surrounding ground surface.

b. When loading tests are made on bearing materials of classes 10 to 13, inclusive, suitable methods shall be used to prevent evaporation from the materials being tested.

c. A load test shall be applied which will produce a unit pressure equal to that for which the proposed foundations are designed. This load shall be allowed to remain undisturbed until no measurable settlement occurs during a period of 24 hr. The load shall then be doubled in increments not exceeding 25 per cent of the design load. At least 4 hr. shall elapse between the application of successive increments. The total load shall be allowed to remain undisturbed until no measurable settlement occurs during a period of 24 hr.

d. Measurement of settlement shall be accurate to $\frac{1}{32}$ in. and shall be taken and recorded every hour during the first 6 hr. after the application of each increment, and at least once every 12 hr. thereafter.

e. When the design load upon bearing materials of classes 1 to 10, inclusive, causes settlement of less than $\frac{3}{8}$ in. and twice the design load causes settlement of less than 1 in., the design load shall be allowed; but if medium or soft clay underlies these materials the vertical pressure in such clay shall not exceed that allowed in Art. 1-15.

f. Whenever the proposed foundation rests on or is underlain by bearing materials of classes 11 to 13, inclusive, the results of loading tests must be interpreted in conjunction with accurate soil profiles showing magnitude and variation in the thickness of these strata. If this information, in the opinion of the commissioner, is not sufficient to determine whether the design load will cause excessive settlement, as might occur due to a thick stratum of clay, or dangerous differential settlement, as might occur when the underlying clay stratum varies considerably in thickness, the commissioner may require an analysis to be made of the probable magnitude, rate, and distribution of settlement of the proposed structure, based on:

(1) A study of settlement records of near-by structures having essentially the same foundation conditions.

(2) Consolidation tests and other investigations of undisturbed samples of the compressible materials.

CHAPTER II

SOME FUNDAMENTALS OF SOIL MECHANICS

2-1. Laboratory Soil Tests. Foundation problems are of such a character that a strictly theoretical mathematical treatment will always be impossible. Experience must be the ultimate guide to their solution. However, developments in the field of soil mechanics during the past decade brighten the prospects of foundation design becoming an applied or semiempirical science. The elastic and plastic properties of soils are gradually becoming better understood. This is also true of the laws governing stress distribution throughout soil masses. The pressing need at present is the accumulation of observations on soil behavior under field conditions to the end that proper coefficients may be applied to mathematical formulas. Only by a combination of mathematical analyses, laboratory tests, and empirical data gathered on the field can foundation engineering become an applied science.

The four basic laboratory soil tests of interest to the foundation engineer are (*a*) the laterally confined consolidation test, (*b*) the permeability test, (*c*) the direct shearing test, and (*d*) the unconfined or partially confined compression test. The consolidation test is perhaps the most important one of the four, for on it we rely for information relative to the probable amount of settlement of our structure. This test also, in conjunction with the permeability test, gives us the probable rate of settlement. The shearing test and the unconfined or partially confined compression test directly measure the strength of the soil.

The type of apparatus used in the consolidation test is illustrated in Fig. 2-1*a*. It consists essentially of a heavy brass ring within which the specimen fits snugly and is confined laterally; a base plate which holds the ring in position; and porous stones and drainage channels, which drain the sample, top and bottom. The load is applied through a lever arrangement, and the deformation is measured by means of an Ames dial. Loads are applied in definite increments, each load being allowed to act until complete consolidation under that load has been effected, as indicated by the dial reading. On completion of the load increments, the entire load is removed and the resulting expansion measured.

The success of this test is measured largely by the proper preparation of the sample, which must fit the ring exactly and be fully confined by it. The sample must have parallel faces, and the thickness must be carefully measured. In preparing the sample great care must be exercised to disturb its structure as little as possible.

The time-compression curve resulting from the consolidation test gives a measure of the impermeability of the specimen. As a check on this, the consolidation apparatus may be transformed into a permeameter by filling a tube, which connects with the lower porous plate, with water and observing the time required to lower

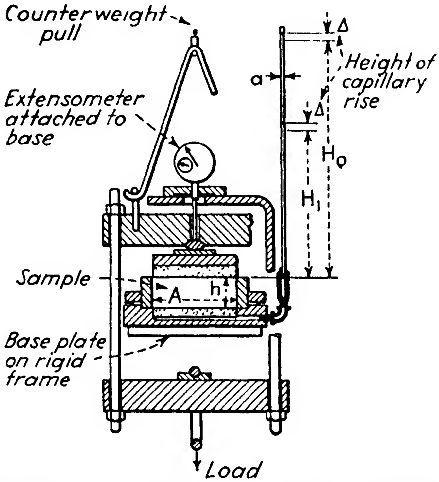


FIG. 2-1*a*.—Consolidation Test Apparatus. (*Courtesy of Boston Soc. Civil Engrs.*)

the elevation of the water in the tube by the water flowing through the specimen and out through the upper porous plate.

Figure 2-1*b* illustrates a typical apparatus for testing directly the shearing strength of uncemented soils.[1] It consists of two sections, between which the sample is placed. The lower section is held rigidly on a testing bench, with provision for mounting extensometers to measure the vertical and horizontal motions of the upper section. The upper section consists of a frame and a movable piston, the latter often being a porous stone to allow free drainage as consolidation of the sample takes place. Just prior to applying the horizontal force, the upper frame is raised and locked in position to permit the entire vertical load to be carried by the soil. The

[1] Rutledge, Philip C., Recent Developments in Soil Testing Apparatus, *J. Boston Soc. Civil Engrs.*, vol. 22, p. 227, October, 1935.

horizontal load is applied through the upper frame in the plane of separation between the two sections. Gratings of various designs are provided to distribute the shearing stress as uniformly as possible and to ensure failure in the plane of the horizontal load.

The unconfined compression or triaxial test is the most satisfactory means of measuring the shearing strength of cemented sands and of clays in a solid form. It is analogous to the compression test of concrete cylinders and is made in the same manner. It is sometimes necessary to band the specimen near the top and bottom to prevent local failures near the ends. As the strength of the specimen is greatly affected by changes in moisture content,

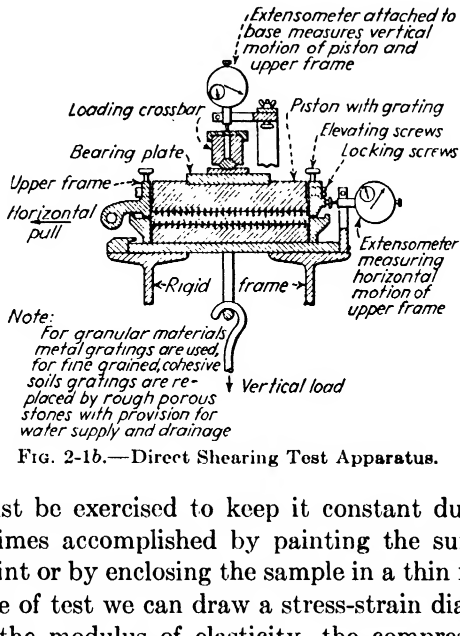


FIG. 2-1b.—Direct Shearing Test Apparatus.

great care must be exercised to keep it constant during the test. This is sometimes accomplished by painting the surface with an impervious paint or by enclosing the sample in a thin rubber casing. From this type of test we can draw a stress-strain diagram, as well as determine the modulus of elasticity, the compressive strength (from which the shearing intensity is found), and the type of failure (rupture or plastic flow).

The compression or triaxial test for shearing strength can also be made on cohesionless soils such as sands. This is made possible by enclosing a cylindrical specimen in a rubber envelope and subjecting the specimen to an external fluid pressure as the vertical load is applied. The effect of this fluid pressure is analogous to the pressure of cohesion and gives to the granular mass the characteristics of a solid.

Figure 2-1*c* illustrates one form of testing apparatus.[1] The sample (10), enclosed in a rubber envelope (9), rests on a porous disk fitted into the pedestal of the base (12). The cylindrical compression chamber is enclosed by a wall of Lucite (6), soft rubber gaskets (11) being provided at both ends. A $\frac{3}{4}$-in. steel piston rod (1) passes through the center of the circular head (3) and transmits the vertical load to the specimen. The contact between the piston and the cap (5) is designed so that it will not exclude the hydrostatic pressure over the area underneath the piston rod. To accomplish this a steel hemisphere is seated in the cap, and a conical counterbore is made in the piston rod to admit fluid pressure to the counterbore. This results in a circular line bearing between the piston and the head. The porous disks in the base and in the cap connect with counterbores, which in turn connect with outside piping for vacuum and moisture controls. Lateral pressure is produced by the use of glycerin rather than water in order to reduce piston friction and leakage around the piston.

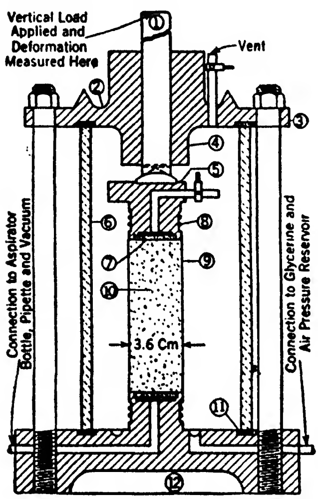


FIG. 2-1*c*.—Sectional Sketch of Triaxial Compression Chamber.

In preparing a sample of sand for testing, a rubber tube is first bound to the boss of base pedestal, after which a metal forming shell is placed in position, extending down over the boss. This shell is necessary to mold a cylindrical specimen of soil and to hold it in shape until a vacuum can be applied. A known weight of dry sand is then placed inside the rubber tube and tamped to the desired density. The rubber tube is turned up around the lower boss of the cap and fastened. A vacuum is applied at the base of the sample to give rigidity, as this makes it easier to fasten the tube. A vacuum is then applied at the top. After a few minutes the vacuum at the bottom is discontinued and a petcock opened to permit water to fill all voids in the sand. Following this, a vacuum is reestablished at the bottom. The forming shell may now be removed, as the sample will stand erect. The petcock in the top vacuum is next closed and the pipe leading to the outside removed.

[1] Watson, John D., The Technique of Triaxial Compression Tests, *Civil Eng.*, vol. 9, p. 731, December, 1939.

After the external dimensions of the cylindrical sample have been measured, the compression chamber is placed in position and the glycerin forced in until the desired pressure has been reached. The test data consist of observations on loads, vertical deformations, and volume changes. Deformations are measured by means of an extensometer attached at the top of the piston. Volume changes are measured by the rise and fall of the water level in a pipette which is connected with the specimen through the counterbores in the base.

2-2. Cohesionless-soil Consolidation. Soil is a granular material composed of a heterogeneous mass of individual particles. Its stability under loads depends not only on the physical characteristics of the individual particles, such as shape, size, and variation of size, but also on the degree of compaction and the water content.

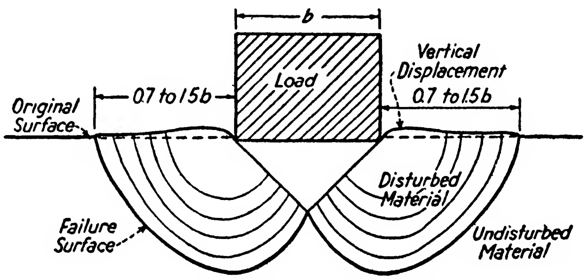


FIG. 2-2a.—Diagram showing Failure of Granular Material Beneath a Bearing Plate.

A structure resting on a soil settles because of (*a*) soil consolidation (reduction of voids) and elastic compression and (*b*) lateral displacement of the soil. The settlement of a structure with a shallow foundation[1] is illustrated by Fig. 2-2*a*. The wedge of earth directly under the load is compressed and moves downward, while the material outside of the wedge moves downward under compression, as well as outward and upward under shearing action along surfaces on which the ratio of shearing force to shearing resistance is a maximum.

That portion of the settlement due to soil consolidation is limited to the possible increase in density by a reduction of voids, for the stresses do not usually attain a value where the elastic deformations of the soil particles themselves are of any consequence or where the soil particles are crushed. In coarse-grained soils it is very difficult to effect any considerable amount of consolidation by bearing pressure alone, except when the grains are in a very loose

[1] *Proc. A.S.T.M.*, vol. 35, Part 2, p. 491, 1935.

state. However, compression accompanied by vibrating disturbance, or vibration alone, may result in a substantial amount of consolidation. The finer the soil the greater the possible consolidation.

Well-graded bank sand composed of round particles may have a percentage of voids ranging from 27 to 44,[1] or from 37 to 79 when expressed in terms of the volume of solid sand. Plum Island (Massachusetts) beach sand (which has angular grains, an effective size of 0.54 mm., and a uniformity coefficient of 1.70) has a percentage of voids varying from 38 in the densest state to 46 in the loosest state,[2] or from 61 to 85 when expressed in terms of solid sand.

Figure 2-2*b* shows the results of a consolidation test on very loose beach sand,[3] the ordinates representing the void ratio—ratio of volume of voids to volume of solid material—and the abscissas representing the pressure in kilograms per square centimeter. When compressed in the loose state, the material followed the curve *AB* and rebounded to point *C* on release of load. At point *C* the sample was disturbed by a light hammer blow on the bottom of the container. This resulted in a consolidation from *C* to *D*. On reapplication of load, *DE* represented the load consolidation curve. Further cycles of load release, tapping, and reapplication of load indicate that, as the soil becomes denser, the load-consolidation curve becomes flatter. From this test it appears that, for any degree of density beyond the very loosest state, there is set up a soil structure having considerable rigidity which cannot be destroyed by a compression force alone; when, however, a vibratory action or remolding effect is introduced, a marked consolidation results.

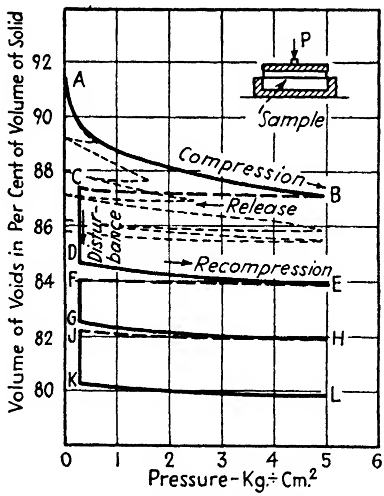


FIG. 2-2*b*.—Consolidation Test on Loose Sand under Repeated Loads.

Thus we see that in a cohesionless soil there is no direct relationship between the compressive stress and the degree of consolidation,

[1] Taylor and Thompson, "Concrete, Plain and Reinforced," 3d ed., p. 138.

[2] *Eng. News-Record*, vol. 112, p. 136, Feb. 1, 1934.

[3] Shearing Resistance of Soils, *J. Boston Soc. Civil Eng.*, vol. 21, p. 242, July, 1933.

for the latter depends more on the previous history of the sample than on the present load. For example, in the case of the sand of Fig. 2-2*b*, for a pressure of 5 kg. per sq. cm. the void ratio is 0.87 (point *B*) if we start with a loose sand (void ratio about 0.91). However, if the sample is vibrated to a void ratio of slightly over 0.80 (point *K*) prior to load application, a pressure of 5 kg. per sq. cm. corresponds to a void ratio of slightly less than 0.80 (point *L*).

Soil consolidation never results in a foundation failure but only in excessive settlement. A decrease in the percentage of voids from 44 to 27 will result in a decrease of 23 per cent in the thickness of the stratum. Where complete failure—continuous settlement at a rapid rate—occurs, it is due to shear and not to consolidation. In general, only shallow foundations with relatively small bearing areas are liable to shear failure. For large bearing areas, particularly at some depth and in plastic soils, the safe loading will be limited by permissible settlement rather than by soil strength.

In studying the effect of shape, size, and variation in size of sand grains, tests show that a well-graded concrete sand having angular-shaped particles has a bearing strength of over four times that of standard Ottawa sand, in which the grains are more rounded and the variation in grain size is small. These tests were made on vibrated samples and only a portion of the surface was loaded.

2-3. Shearing Resistance of Cohesionless Soils. The shearing resistance of a soil depends on two factors, (*a*) the normal stress and (*b*) the coefficient of friction, and equals the product of the two. The normal stress may be due to the presence of external loads or to internal cohesion and surface tension of the included moisture. In fine sands, cohesion is absent, and in coarse sands both cohesion and surface tension are missing. The coefficient of friction depends on the character and degree of consolidation of the soil particles. The effect of density of the soil mass on the shearing resistance[1] is shown in Fig. 2-3*a*, which gives the results of shearing tests on cohesionless standard Ottawa sand. Note that for equal normal loads the shearing resistance of dense sand is almost double that of loose sand.

The influence of grain type is illustrated by the following shear-test results taken from the same source as Fig. 2-3*a*, all values being expressed in pounds per square inch, the normal intensity being 20: chilled No. 12 shot, 19; standard Ottawa sand, 27; sharp concrete sand graded to standard Ottawa size, 42; and well-graded sharp concrete sand, 45. All materials were vibrated to maximum

[1] *Proc. A.S.T.M.*, vol. 35, Part 2, p. 499, 1935.

compaction. These figures indicate that the shape of the sand grains exercises a greater influence than does gradation of size. These tests, as well as those shown in Fig. 2-3*a*, indicate that the coefficient of friction is essentially constant for any given sand of constant density.

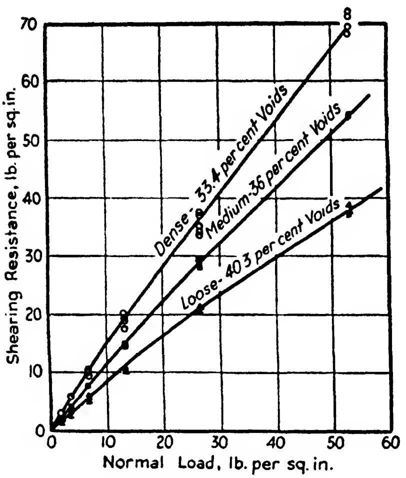


FIG. 2-3*a*.—Effect of Density on Shearing Resistance.

An interesting phenomenon in connection with shear tests of cohesionless soils is that of volume change under shearing stress action. The volume increases in the case of a dense sand and decreases in the case of a loose sand. In Fig. 2-3*b* the upper left-hand illustration shows a dense sand subjected to a normal force P; in the upper right-hand illustration a shearing force S has been added. In the dense sand the grains are so closely interlocked that shearing displacement is not possible with-

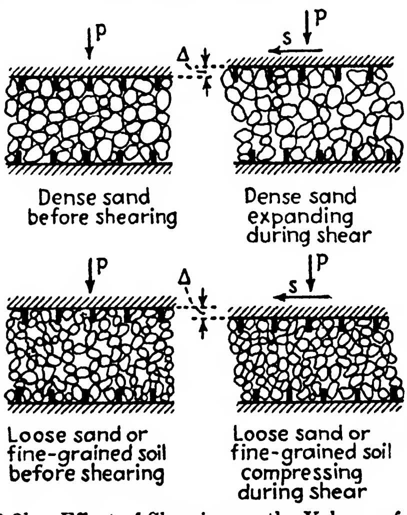


FIG. 2-3*b*.—Effect of Shearing on the Volume of Soils.

out loosening the grain structure somewhat. On the other hand, if we start with a loose sand, the effect of lateral displacement is to reduce the voids as shown in the lower figures.

The curves of Fig. 2-3*c* graphically show the relationships between lateral displacement, shearing resistance, and soil-density change for any constant normal force.[1] As a shearing force is applied to a dense sand to cause a lateral displacement, the shearing resistance increases as shown by the line *AB* until a maximum (S_p) is reached. With further displacement the resistance decreases along the line *BC* to S_L and thereafter remains constant. While the shearing resistance is increasing along the line *AB*, the soil density is decreasing along the line *DE*. Likewise, while the resistance is decreasing along *BC*, the density is decreasing along *EG* to *G*, at which point it becomes constant.

If we start with a very loose sand, the resistance increases with lateral displacement as shown by *AC*, and at the same time the density is increasing along the line *FG*. For very fine-grain soils like silt and clay the volume usually decreases during a shearing test under normal pressure, regardless of whether it was originally in a dense or loose state.

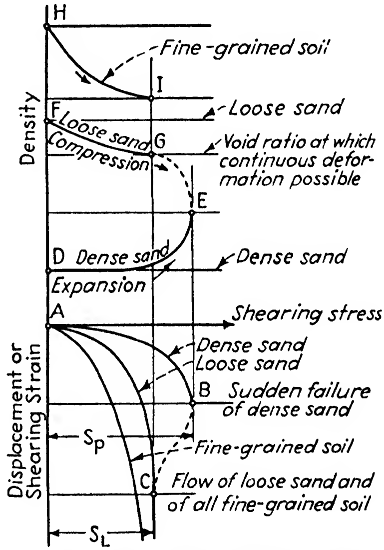


FIG. 2-3c.—Relationships between Lateral Displacement, Shearing Resistance, and Soil Density.

The density at which the volume and shearing resistance remain constant as further lateral displacement takes place (point *G*) is called the "critical density." The density of fine and medium uniform-size sands in a loose state is generally considerably less than this critical density, whereas many coarse-grained and well-graded mixtures of cohesionless soils are at about the critical density when in their loose state. A loose sand may be made more dense in one of the following three ways: (*a*) vibrating the mass, (*b*) giving the soil a lateral movement, and (*c*) applying a normal load. Vibrating is the most effective method and the only one by which a maximum density is possible. Lateral motion accompanied by a direct pressure is effective, but direct pressure alone will not produce a high density.

[1] Characteristics of Cohesionless Soils Affecting the Stability of Slopes and Earth Fills, *J. Boston Soc. Civil Engrs.*, vol. 23, p. 13, January, 1936.

If the voids in a mass of loose sand are filled with water, some of this water must be forced out before any reduction of volume can take place. If there is a considerable resistance to movement of the water, there will be set up in the sand mass a considerable hydrodynamic pressure when the sand is subjected to lateral shear displacement. Under this condition some of the normal pressure between sand grains will be transferred to the water, and, as the water has no shearing resistance, the shearing resistance of the sand mass may be greatly reduced. In the case of a sand having a density greater than the critical density, on application of lateral deformation the voids tend to increase, and so for a saturated sand there will be temporarily created tension in the water and a corresponding increase in pressure between the grains. Hence deformation of a dense, saturated sand may result in a shearing resistance higher than the normal value.

From the above discussion it is evident that the engineer may count on a coefficient of friction for dry sand of only $\frac{S_L}{P}$. If he is dealing with a loose sand, S_L represents the maximum obtainable resistance. On the other hand, if the sand is dense, for a small lateral displacement he may expect a larger value (S_p), but with further displacement this will reduce to S_L. If the sand is loose and carries a large amount of water, the shearing resistance may be much less than S_L, but for a saturated, dense sand it will be greater.

Cohesionless soils in a state of less than the critical density are sometimes compacted and stabilized to a considerable depth and over large areas by special vibration machinery. By this means settlement under loads is decreased and the shearing resistance is increased when the soil is water bearing.

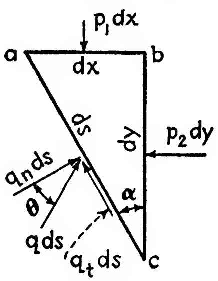


FIG. 2-4a.

2-4. The Mohr Diagram. Let Fig. 2-4*a* represent an element of unity thickness perpendicular to the paper, one face being vertical, one horizontal, and one making the angle α with the vertical. Let the stress on *ab* be vertical and have an intensity of p_1. The stress on *bc* will be horizontal since the shearing intensities on any two planes at right angles to each other are equal. Let p_2 be the intensity of stress on *bc* and q that on *ac*, its obliquity being θ and its two components q_n and q_t.

The three forces $p_1\,dx$, $p_2\,dy$, and $q\,ds$ form a balanced system of forces and so graphically must form a closed triangle (Fig. 2-4*b*),

hence

$$q^2\,ds^2 = p_1^2\,dx^2 + p_2^2\,dy^2,$$

$$q = \sqrt{p_1^2 \sin^2 \alpha + p_2^2 \cos^2 \alpha},$$

and

$$\tan\,(\alpha + \theta) = \frac{p_1\,dx}{p_2\,dy} = \frac{p_1}{p_2}\tan \alpha.$$

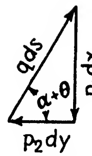


FIG. 2-4b.

A graphical construction for finding q in terms of $(p_1 + p_2)/2$ and $(p_1 - p_2)/2$ is as follows: Through the point O of Fig. 2-4c lay off on a line normal to the plane ac the length $Om = (p_1 + p_2)/2$; then lay off $mr = (p_1 - p_2)/2$, this line making the same angle α with the horizontal as Om. If Om and mr are resolved into vertical

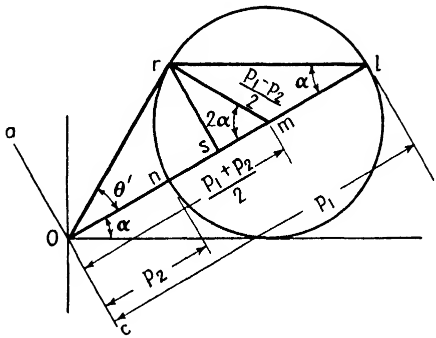


FIG. 2-4c.

and horizontal components, it can be shown that the vertical component of Or is $p_1 \sin\,\alpha$ and the horizontal component is $p_2 \cos\,\alpha$, therefore

$$Or = \sqrt{p_1^2 \sin^2 \alpha + p_2^2 \cos^2 \alpha}, \qquad \text{hence} \qquad Or = q,$$

and

$$\tan\,(\alpha + \theta') = \frac{p_1 \sin \alpha}{p_2 \cos \alpha} = \frac{p_1}{p_2}\tan \alpha, \qquad \text{hence} \qquad \theta' = \theta.$$

If rs is drawn normal to the line Om, then

$$Os = q_n = \frac{p_1 + p_2}{2} - \frac{p_1 - p_2}{2}\cos 2\alpha,$$

and

$$rs = q_t = \frac{p_1 - p_2}{2}\sin 2\alpha.$$

It will be noted that Om and mr are independent of the angle α. Draw a circle with a radius of mr and its center at m. Then

$$Ol = Om + ml = \frac{p_1 + p_2}{2} + \frac{p_1 - p_2}{2} = p_1 \text{ and}$$

$$On = \frac{p_1 + p_2}{2} - \frac{p_1 - p_2}{2} = p_2.$$

It will also be noted that the angle $nmr = 2\alpha$, and $mlr = \alpha$. Hence it is evident that this figure may be used to find the intensity of stress on any plane. A line from O to any point r' on the circle gives the intensity of stress on some plane, the angle this plane makes with the vertical being mlr'. This is known as the Mohr circle and in constructing the same the base line Ol is usually made horizontal.

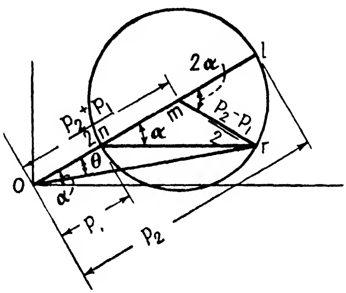


FIG. 2-4d.

Where p_2 is large compared with p_1, the obliquity of the stress qds of Fig. 2-4a will be above the normal. To find q graphically in terms of $(p_2 + p_1)/2$ and $(p_2 - p_1)/2$, through the point O of Fig. 2-4d, lay off Om as we did in Fig. 2-4c. In laying off $mr = (p_2 - p_1)/2$, we must draw it from m downward to the right, instead of upward to the left. Then $Or = q$. If we draw a circle with its center at m and with a radius of mr, then $Ol = p_2$ and $On = p_1$. Also note that the angle $lmr = 2\alpha$ and $mnr = \alpha$.

2-5. Shearing Resistance of Cohesionless Soils from Triaxial Tests. As noted in Art. 2-1, in testing for the shearing resistance of cohesionless soils by the compression or triaxial test, the test cylinder is enclosed in a rubber envelope and subjected to a fluid pressure while the axial load is being applied. The stress conditions will be similar to that shown in Fig. 2-4a, where $p_1 = p_a + p_2$, in which p_a is the intensity of applied axial load and p_2 is the intensity of fluid pressure. On some plane such as ac the obliquity of stress (θ) will be greater than on any other plane, and, if p_a represents the axial intensity of applied load at which the specimen fails, this obliquity will equal ϕ, where ϕ is the angle whose tangent equals the coefficient of friction of the sand.

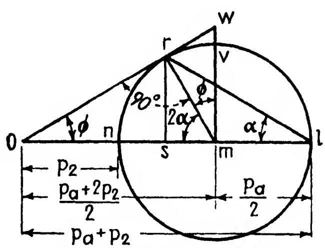


FIG. 2-5a.—Mohr Circle for the Triaxial Test.

In Fig. 2-4c it is evident that the angle of obliquity $\theta' = \theta$ will be a maximum when Or is tangent to the Mohr circle, in which

$Orm = 90$ deg., hence

$$\sin \phi = \frac{\frac{p_1 - p_2}{2}}{\frac{p_1 + p_2}{2}} = \frac{p_1 - p_2}{p_1 + p_2} = \frac{p_a}{p_a + 2p_2}. \tag{2-5a}$$

The Mohr circle for the conditions that $p_1 = p_a + p_2$ and the maximum obliquity of stress equals ϕ is shown in Fig. 2-5*a*, a horizontal axis being used for *Ol*. From this figure it is evident that $2\alpha = 90$ deg. $- \phi$, or $\alpha = 45$ deg. $- \frac{\phi}{2}$; hence the plane on which shearing failure occurs makes an angle of 45 deg. $- \frac{\phi}{2}$ with the vertical. The intensity of stress on this plane is *Or*, the tangential intensity *rs*, and the normal intensity *Os*. Here $rs = rm \cos \phi = \frac{p_a}{2} \cos \phi$, and $Os = rs \cot \phi = \frac{p_a}{2} \cos \phi \cot \phi$. It will be observed that the maximum unit shearing stress $vm = p_a/2$ occurs on a plane where $\alpha = 45$ deg. However, failure does not take place on this plane, for here the resistance is $wm = \frac{p_a + 2p_2}{2} \tan \phi$, which exceeds the shearing intensity by the amount *wv*.

The value of ϕ may be determined from Eq. 2-5*a*, after finding from a test the simultaneous values of p_a and p_2 at which the sand specimen fails. A graphical solution for ϕ is to lay off on Fig. 2-5*a*, $Om = (p_a + 2p_2)/2$ and then draw a circle with *m* as a center and with a radius of $p_a/2$. A line is then draw from *O* tangent to this circle, and $mOr = \phi$.

Results of tests made on two samples of silty fine sand are given by Watson,[1] as follows:

	Loose sample	Dense sample
Volume solids	45.7 cc.	62.0 cc.
Volume voids at start	37.6 cc. = 82.2%	37.8 cc. = 61%
Volume voids at failure	34.6 cc. = 75.5%	45.36 cc. = 72.8%
Lateral pressure, p_2	4.50 kg. per sq. cm.	0.84 kg. per sq. cm.
Axial pressure, $p_1 = p_a + p_2$	16.95 kg. per sq. cm.	4.60 kg. per sq. cm.
$\phi = \sin^{-1} \frac{p_1 - p_2}{p_1 + p_2}$	35°28′	43°44′

[1] Watson, John D., The Technique of Triaxial Compression Tests, *Civil Eng.*, vol. 9, p. 731, December, 1939.

2-6. Rankine's Earth-pressure Theory. In Rankine's theory for the determination of lateral earth pressure, the soil is assumed to be cohesionless, and the stress on any plane parallel to the surface of the ground is assumed to be vertical and to equal the weight of the earth above the plane.

Where the ground surface is horizontal, the intensity of pressure on any horizontal plane distant h (d often used to denote this distance) below ground surface will be $p_1 = wh$, where w is the weight of a unit volume of the earth. The intensity of stress, p_2, on a vertical plane at the same point is assumed to be just large enough to prevent sliding of the particles along that plane ac (Fig. 2-4a) on which a maximum sliding tendency exists. As in Art. 2-5, the obliquity of stress on this plane will be ϕ, this being the angle whose tangent equals the coefficient of friction between the soil particles. As explained in Art. 2-5, by referring to Fig. 2-4c, it is evident that the angle of obliquity $\theta' = \theta$ will be a maximum when Or is tangent to the Mohr circle, in which case $Orm = 90$ degrees. If we denote the value of p_2 that makes this obliquity ϕ as p_{2_a}, we have

$$\sin \phi = \frac{\frac{p_1 - p_{2_a}}{2}}{\frac{p_1 + p_{2_a}}{2}},$$

$$p_{2_a} = p_1 \frac{1 - \sin \phi}{1 + \sin \phi} = wh \frac{1 - \sin \phi}{1 + \sin \phi} = wh \tan^2 \left(45 \text{ deg.} - \frac{\phi}{2}\right). \quad (2\text{-}6a)$$

But p_{2_a} is not the only value of p_2 that will hold the earth in equilibrium. As p_2 is increased from its minimum value of p_{2_a}, equilibrium is constantly maintained until the obliquity of stress on some plane ac—on which the obliquity is a maximum—becomes ϕ on the upper side of the normal. At this point upward movement of the earth mass will result. Here $p_2 = p_{2_p}$ is greater than p_1, and so $mr = (p_{2_p} - p_1)/2$. Hence

$$\sin \phi = \frac{\frac{p_{2_p} - p_1}{2}}{\frac{p_{2_p} + p_1}{2}},$$

$$p_{2_p} = p_1 \frac{1 + \sin \phi}{1 - \sin \phi} = wh \frac{1 + \sin \phi}{1 - \sin \phi} = wh \tan^2 \left(45 \text{ deg.} + \frac{\phi}{2}\right). \quad (2\text{-}6b)$$

In design work p_{2_a} is called the "active earth pressure," and it represents the minimum lateral intensity of stress with which equilibrium is possible. It is the minimum unit lateral pressure that would be exerted by the earth on a retaining wall having a vertical back face. Likewise p_{2_p} is called the "passive earth pressure," and it represents the maximum lateral intensity of stress that a retaining wall could exert on the earth without causing an upward movement of the earth mass.

Given p_1 and the angle ϕ, the following is a simple graphical solution for finding p_{2_a}: In Fig. 2-6*a* lay off to any scale $Ol = p_1$, and from O draw Or making the angle ϕ with Ol. Draw the circle

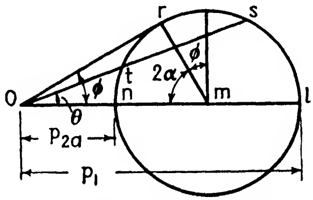


FIG. 2-6*a*.—Mohr Diagram for Active Earth Pressure in Cohesionless Soils.

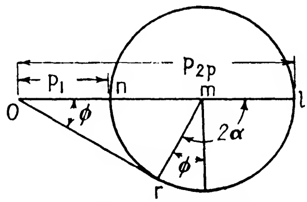


FIG. 2-6*b*.—Mohr Diagram for Passive Earth Pressure in Cohesionless Soils.

which has its center on Ol, passes through l, and is tangent to Or. Then $On = p_{2_a}$. It will be observed that $2\alpha = 90$ deg. $- \phi$, or $\alpha = 45$ deg. $- \dfrac{\phi}{2}$; hence the plane on which sliding takes place makes an angle of 45 deg. $- \dfrac{\phi}{2}$ with the vertical.

To find p_{2_p} graphically, lay off (Fig. 2-6*b*) $On = p_1$, and from O draw Or, making the angle ϕ with On. Then draw the circle which has its center on On produced, passes through n, and is tangent to Or. Then $Ol = p_{2_p}$. It will be observed that $2\alpha = 90$ deg. $+ \phi$, or $\alpha = 45$ deg. $+ \dfrac{\phi}{2}$; hence the plane on which sliding takes place makes an angle of 45 deg. $+ \dfrac{\phi}{2}$ with the vertical.

Rankine's theory is widely used to determine the active earth pressure on the back of retaining walls. Let us consider a wall h_1 ft. high, the back of the wall being vertical and the surface of the ground horizontal. In Fig. 2-6*a*, Ol (to any scale) represents the intensity of stress on a horizontal plane at some point in the ground, and to the same scale On represents the intensity of stress on a vertical plane at the same point. This point is at a depth h, where $Ol = wh$, or $h = Ol/w$. Therefore the lateral intensity of

stress at a depth of h_1 will be $On\frac{h_1}{h} = \frac{On}{Ol}wh_1$. The total earth pressure for a height of h_1 and unity length of wall is $\frac{1}{2}w\frac{On}{Ol}h_1^2$.

Where the ground surface is not horizontal but slopes upward away from the wall at an angle of θ with the horizontal, we proceed as follows: In Fig. 2-6*a* lay off Os, making the angle θ with Ol. To any scale, Os represents the intensity of stress at some point in the ground on a plane parallel to the surface, because of the Rankine assumption that the stress on any plane parallel to the surface is vertical. To the same scale, Ot represents the intensity of stress on a vertical plane, since the obliquity of stress on a vertical plane equals that on a plane parallel to the surface. Hence it follows that the total lateral stress on the back of the wall equals $\frac{1}{2}\frac{wOt}{Os}\cos\theta h_1^2$ and acts parallel to the surface.

Where the back face of the wall is not vertical, we first get the total stress on a vertical plane and then combine it with the weight of the wedge between this plane and the back of the wall.

The following formulas give the active earth pressures per horizontal foot on a vertical plane for a height of h_1 ft. where the ground surface is horizontal, w representing the weight of a cubic foot of earth.

For no surface loading

$$P_1 = \frac{1}{2}wh_1^2\tan^2\left(45\text{ deg.} - \frac{\phi}{2}\right), \tag{2-6c}$$

and where the surface loading equals w_1 lb. per sq. ft.

$$P_2 = \left(\frac{1}{2}wh_1^2 + w_1h_1\right)\tan^2\left(45\text{ deg.} - \frac{\phi}{2}\right) \tag{2-6d}$$

Likewise the total passive earth pressure is

For no surface loading

$$P_3 = \frac{1}{2}wh_1^2\tan^2\left(45\text{ deg} + \frac{\phi}{2}\right), \tag{2-6e}$$

and where the surface loading equals w_1 lb. per sq. ft.

$$P_4 = \left(\frac{1}{2}wh_1^2 + w_1h_1\right)\tan^2\left(45\text{ deg.} + \frac{\phi}{2}\right). \tag{2-6f}$$

The value of w depends on the type of soil and its degree of compaction. For values widely used see Table 6 of "Carnegie Steel Sheet Piling" (1931). As discussed in Art. 2-3 values of ϕ

vary between wide limits, depending on the soil structure, degree of consolidation, and water content. Values of from 35 to 40 deg. are widely used for average conditions. The table referred to above gives values of ϕ for a considerable number of granular materials.

2-7. Plastic Soils. The behavior of plastic soils, such as clay and silt, is quite different from that of cohesionless sand. In the natural state, their structure is much more open, as shown by their larger void ratio; hence deformation under compression loads is many times that of sand. However, this consolidation may take place at a very slow rate, since it can develop only by forcing out the water in the voids. Owing to the high degree of impermeability of most clays, this is a very slow process, especially for thick beds. This explains why buildings resting on clay often continue to settle for many years, and it also shows why a short-time loading test is of little value in plastic soils. The time required to effect a given percentage of maximum consolidation for any load intensity varies as the square of the thickness of the bed. For example, if a $\frac{1}{2}$-in. thickness reaches 90 per cent consolidation in 10 hr. for a given load, then a soil layer 10 ft. thick would require 66 years to reach the same degree of consolidation for the given load.

In plastic soils the shearing resistance is developed through the presence of cohesion and surface tension of the enclosed moisture, as well as by normal stresses due to external loads. Cohesion is caused by the molecular attraction of soil particles. In coarse-grained soils both cohesion and surface tension are absent, but in fine-grained soils molecular attraction is of the order of the weight of the particles themselves. Surface tension also increases as grain size decreases but disappears entirely when the soil mass is saturated with water.

In describing clays certain terms are used to denote plasticity conditions. The "liquid limit" is defined as that moisture content, expressed as a percentage of the weight of oven-dry soil, at which the soil will just begin to flow when lightly jarred ten times. The "plastic limit" is the lowest moisture content, expressed as a percentage of the weight of the oven-dry soil, at which the soil can be rolled into threads $\frac{1}{8}$ in. in diameter, without the threads breaking into pieces. "Plasticity index" is defined as the difference between the liquid limit and the plastic limit. The "shrinkage" limit of a soil is that moisture content, expressed as a percentage of the weight of the oven-dry soil, at which a reduction of moisture content will not cause a decrease in the volume of the soil mass but at which an increase in moisture content will cause an increase in the volume of the soil mass. Standard methods of testing for

these properties of soil may be found in the publications of the American Society for Testing Materials.

Soils at the liquid limit have practically no cohesion and only a small shear resistance. The plastic limit represents the condition below which the physical properties of the contained water are no longer identical with those of free water. It is also the point at which a very slight increase in the water content results in a very rapid increase in the compression deformation under loads. The plasticity index indicates the increase in water content necessary to reduce the cohesion between the particles to zero by increasing the thickness of the water film that separates them. The shrinkage limit represents the point at which the soil passes from a semi-solid to a solid state. At this point the soil changes from a dark to a light color.

At the liquid limit there is practically no capillary pressure, whereas at the plastic limit it amounts to several atmospheres. As evaporation proceeds, there comes a point where the force required to produce a volume change equal to the volume of the evaporated water is greater than the maximum possible value of capillary pressure; hence the surface of the capillary water withdraws into the interior of the soil. Air then enters the voids, and we have the shrinkage limit.

2-8. Consolidation Tests of Plastic Soils. In the geological processes of soil building, nature sets up a soil structure that is porous and open and yet strong and rigid. Once this natural structure is disturbed, it cannot be recreated. This phenomenon is illustrated in Fig. 2-8*a*, which presents the results of consolidation tests made on Laurentian clay,[1] which in its natural state is rather rigid, having a cube strength of from 30 to 70 lb. per sq. in. When an undisturbed sample is remolded, it will first break and crumble between the fingers, but with increasing pressure it will suddenly yield and become plastic and sticky.

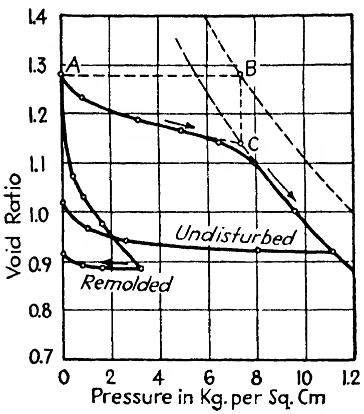


FIG. 2-8*a*.—Consolidation Tests on Laurentian Clay.

Both the undisturbed and remolded specimens had a void ratio of about 1.28 at the beginning of the test. With a load of 2 kg.

[1] Casagrande, Arthur, The Structure of Clay and Its Importance in Foundation Engineering, *J. Boston Soc. Civil Engrs.*, vol. 19, p. 168, April, 1932.

per sq. cm. (4,100 lb. per sq. ft.), the void ratio was 1.2 for the undisturbed specimen and only 0.95 for the remolded specimen, this indicating that the deformation of the remolded specimen was four times that of the undisturbed sample.

Another peculiarity is that, when an undisturbed sample is tested to a pressure equal to the weight of the overburden, the consolidation is much greater than that of the material in its natural state. It seems logical to expect the curve AC to be horizontal out to the point B, the abscissa at this point representing the pressure on the specimen when in its original position. It is probable that some curve, similar to the sloping dotted line through B, more nearly represents the stress-consolidation curve of the soil in its natural state than does the test curve.

One explanation of this phenomenon is as follows:[1] During the process of sedimentation the silt and larger clay particles settled out individually, while clay particles of colloidal size first formed flocks and then gradually settled. When these individual grains and flocks reached the bottom, they formed a very loose, honeycomb structure. Consolidation gradually developed under the growing weight of the overlying sediment. The density of the thin layers of flock between the silt grains increased more rapidly than that of the general mass of the flock. This resulted in a structure composed of stable silt grains bonded together with dense clay and with soft clay occupying the rest of the space. Remolding destroys this bond, causing the softer clay to surround the silt particles and resulting in a much weaker soil.

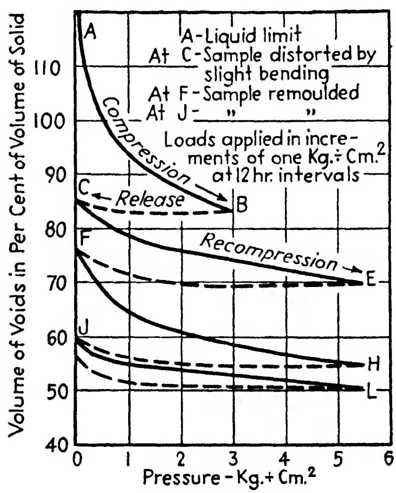


FIG. 2-8b.—Consolidation Test on Clay under Repeated Loads.

When an undisturbed sample is taken from the ground, the external pressure is removed, resulting in an immediate swelling tendency. However, without an increase in water or air content, this expansion is impossible, hence the development of surface tension. The relief of stress in the more highly compressed bond

[1] Casagrande, Arthur, The Structure of Clay and Its Importance in Foundation Engineering, *J. Boston Soc. Civil Engrs.*, vol. 19, p. 168, April, 1932.

clay may cause moisture to be absorbed by the same at the expense of the softer clay, as a result of which the strength of the specimen may be reduced. This may explain why an undisturbed sample suffers consolidation at loads less than the original soil pressure.

Plastic soils subjected to repetitions of loading, release of load, and remolding exhibit the same behavior as sands (Fig. 2-2*b*), but to a much greater degree (Fig. 2-8*b*). For example, with clay the void ratio may be reduced from 1.2 to 0.5 with four- to five-load repetitions.

2-9. Shearing Resistance of Plastic Soils. As stated in Art. 2-7 the shearing resistance of a plastic soil is developed by cohesion and surface tension as well as by the presence of external normal forces. The word "cohesion" is often used to include both the molecular attraction and surface tension, that which is due to molecular attraction being designated as "true cohesion" and that due to capillarity as "apparent cohesion." These cohesive stresses are called "intrinsic stresses," as they are independent of any external loads.

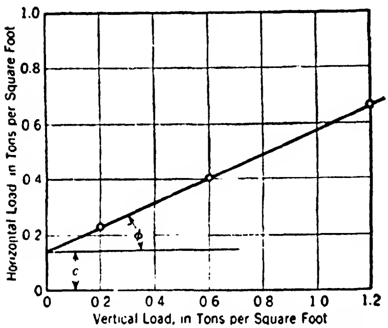


FIG. 2-9*a*.—Shear Diagram of a Plastic Soil.

The shearing strength of a plastic soil may be determined either from a shear test or from a compression or triaxial test. In the shear test the procedure is as outlined in Art. 2-1 for uncemented soils. Figure 2-9*a* illustrates a typical shear diagram for a plastic soil.[1] The ordinate intercept c is the shearing strength due to cohesion; the slope of the curve, essentially a straight line, is ϕ, the angle of internal friction.

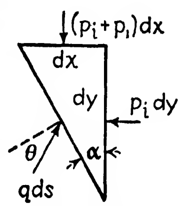


FIG. 2-9*b*.

In finding the shearing resistance by the triaxial test, let Fig. 2-9*b* represent a free body taken from an unconfined test cylinder of plastic soil. On the vertical plane there will be an intrinsic stress intensity of p_i and on the horizontal plane a stress intensity of $p_i + p_1$, where $p_1 = P/A'$, in which P is the ultimate axial load on the test cylinder and A' the mean cross section of the specimen at failure.

Let Fig. 2-9*c* represent the Mohr diagram (Art. 2-4) for this condition, in which $Ol = p_i + p_1$, $On = p_i$, and the angle $lOr = \phi$.

[1] See *Civil Eng.*, vol. 7, p. 570, August, 1937.

Shearing failure will occur on the plane having a maximum obliquity (equal to ϕ), which is where $2\alpha = 90$ deg. $- \phi$, or $\alpha = 45$ deg. $- \frac{\phi}{2}$, α being the angle made by the plane with the vertical. The shear-

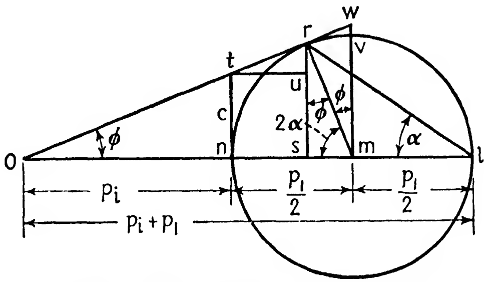


FIG. 2-9c.—Mohr Diagram for Plastic Soil.

ing intensity of stress on the plane of failure is $rs = \frac{p_1}{2} \cos \phi$, while the normal intensity is $Os = p_i + \frac{p_1}{2}(1 - \sin \phi)$. The distance rs also represents the unit shearing resistance of the soil on the plane of failure. This resistance is made up of two parts, (*a*) that due to cohesion and (*b*) that due to sliding friction resulting from the external load. The first is $us = tn$, which is designated as c, and the second is

$$ru = \frac{p_1}{2}(1 - \sin \phi) \tan \phi.$$

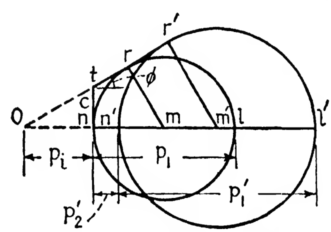


FIG. 2-9*d*.—Mohr Diagrams for Determining Cohesion and Internal Friction.

The maximum shearing intensity is $vm = p_1/2$, which occurs on a 45-deg. plane. On this plane the potential shearing resistance is wm, which exceeds the shearing intensity by the amount wv.

To determine the relative shearing resistances furnished by cohesion [c] and by internal friction $\left[\frac{p_1}{2}(1 - \sin \phi) \tan \phi\right]$, it is necessary to run a partially confined compression test (Art. 2-5) in addition to the unconfined test. Let p'_2 and p'_1 represent, respectively, the unit lateral fluid stress and the unit axial breaking stress for the partially confined test. In Fig. 2-9*d* lay off $nl = p_1$, and draw the Mohr circle as in Fig. 2-9*c*. Lay off $nn' = p'_2$ and $nl' = p'_1$, then draw the Mohr circle through n' and l'. The tangent to these

two circles is called the "Mohr envelope." Its slope with the horizontal gives us the angle ϕ. To get the value of c, prolong $r'r$ until it meets the vertical through n, then $tn = c$. Tests show that the Mohr envelope is not always a straight line; hence a number of partially confined tests should be made to develop fully the curve.

2-10. Effect of Consolidation on Shearing Strength. As in the case of cohesionless soils (Art. 2-3) the degree of consolidation of a plastic soil exerts a large influence on its shearing strength. This is well brought out in some tests[1] made on specimens remolded from just below the liquid limit, the degree of consolidation varying from 0.50 to 4.43 kg. per sq. cm. (1,025 to 9,100 lb. per sq. ft.). The clay had a liquid limit of 41 per cent and a plastic limit of 20.5 per cent.

A cylindrical specimen was first formed in a tube and then removed by a specially designed plunger. The sample was wrapped in filter paper, after which the sides and top were covered with a thin rubber envelope. The bottom of the cylinder rested on a porous disk connected to a drain, so that the excess water was free to escape. Hydrostatic pressure of the desired intensity was then applied all around, except on the base, and maintained until the internal structure of the specimen had become completely adjusted to that pressure, in other words, until thoroughly consolidated for the given cycle. Next it was removed from the pressure chamber and the filter paper taken off to prevent reabsorption of moisture, after which the rubber envelope was again put in place. To guard against further evaporation, the compression tests were run in a room having a relative humidity of nearly 100.

The shearing intensity was calculated from the approximate formula $s = P/2A'$, where P is the ultimate load and A' the average area of cross section at failure. Owing to the large strain, A' is considerably greater than the nominal area of cross section A. If there is no change in volume, $A' = A/(1 - e)$, where e is the unit axial strain. If h denotes the original length of the specimen and h' its deformed length, $A'h' = Ah$. Then

$$A' = \frac{A}{\dfrac{h'}{h}} = \frac{A}{1 - \dfrac{h}{h} + \dfrac{h'}{h}} = \frac{A}{1 - \dfrac{h - h'}{h}} = \frac{A}{1 - e}.$$

The results of these tests are summarized in Fig. 2-10a, one curve showing the relationship between the shearing strength and

[1] Jurgenson, Leo, The Shearing Resistance of Soils, *J. Boston Soc. Civil Engrs.*, vol. 21, p. 242, July, 1933.

the degree of consolidation, while a second curve shows the relationship between the water content and the degree of consolidation. For example, the shearing strength was 0.5 kg. per sq. cm. (1,025 lb.

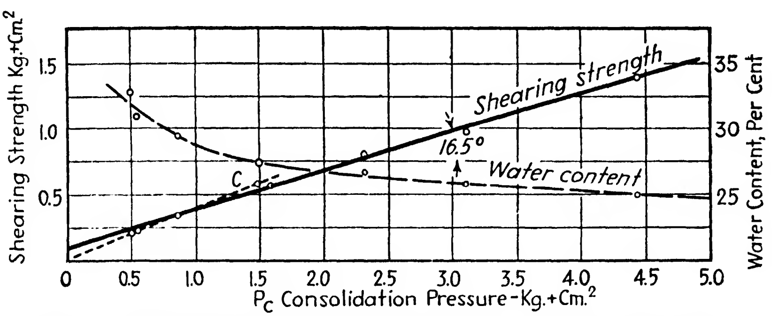


FIG. 2-10a.—Diagram Showing Effect of Consolidation on Shearing Strength of Soil.

per sq. ft.) for a consolidation pressure of 1.5 kg. per sq. cm. (3,070 lb. per sq. ft.). The water content at this degree of consolidation was 27.5 per cent based on the dry weight of the soil.

2-11. Earth-pressure Formulas for Plastic Soils. Formulas for the lateral pressures of cohesionless soils are developed in Art. 2-6. In this article we shall develop corresponding formulas for plastic soils. Let us first develop the formula for active earth pressure, letting p_{2_a} equal the intensity of pressure on a vertical plane and $p_1 = wh$, the intensity of pressure on a horizontal plane. Remembering that we have the intrinsic pressure p_i on both vertical and horizontal planes, we will construct the Mohr diagram (Art. 2-4) by laying off in Fig. 2-11a.

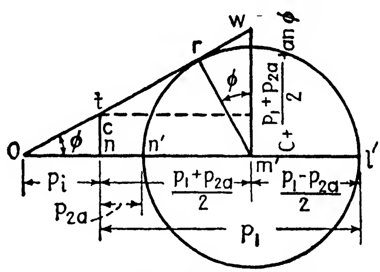


FIG. 2-11a.—Mohr Diagram for Active Earth Pressure in Plastic Soils.

$$Om' = \frac{(p_1 + p_i) + (p_{2_a} + p_i)}{2} = p_i + \frac{p_1 + p_{2_a}}{2}$$

and

$$m'l' = \frac{(p_1 + p_i) - (p_{2_a} + p_i)}{2} = \frac{p_1 - p_{2_a}}{2}.$$

From O draw Or tangent to the circle which has $m'l'$ for its radius. As p_{2_a} is just large enough to prevent the particle of earth comprising our free body from slipping downward, the angle $m'Or$ equals ϕ. Denoting the intrinsic shearing resistance tn as c, we have

$$\cos\phi = \frac{rm'}{wm'} = \frac{\dfrac{p_1 - p_{2_a}}{2}}{c + \dfrac{p_1 + p_{2_a}}{2}\tan\phi}.$$

Solving this equation, we get

$$p_{2_a} = p_1 \frac{1 - \sin\phi}{1 + \sin\phi} - \frac{2c\cos\phi}{1 + \sin\phi}$$

$$= wh\tan^2\left(45 \text{ deg.} - \frac{\phi}{2}\right) - 2c\tan\left(45 \text{ deg.} - \frac{\phi}{2}\right). \qquad (2\text{-}11a)$$

In comparing this formula with that for a cohesionless soil (Eq. 2-6a), we note that the intensity of lateral pressure is decreased by $2c\tan\left(45 \text{ deg.} - \frac{\phi}{2}\right)$.

As explained in Art. 2-6, for passive earth pressure the horizontal intensity of stress is greater than the vertical intensity and so

$m'r = \dfrac{p_{2_p} - p_1}{2}$, hence

$$\cos\phi = \frac{\dfrac{p_{2_p} - p_1}{2}}{c + \dfrac{p_{2_p} + p_1}{2}\tan\phi},$$

and

$$p_{2_p} = p_1 \frac{1 + \sin\phi}{1 - \sin\phi} + \frac{2c\cos\phi}{1 - \sin\phi}$$

$$= wh\cot^2\left(45 \text{ deg.} - \frac{\phi}{2}\right) + 2c\cot\left(45 \text{ deg.} - \frac{\phi}{2}\right). \qquad (2\text{-}11b)$$

In Fig. 2-11a it will be noted that

$$nl' = nm' + m'l' = \frac{p_1 + p_{2_a}}{2} + \frac{p_1 - p_{2_a}}{2} = p_1$$

and $nn' = nm' - m'l' = \dfrac{p_1 + p_{2_a}}{2} - \dfrac{p_1 - p_{2_a}}{2} = p_{2_a}$. This leads to the following simple graphical solution for finding p_{2_a} when c, ϕ, and p_1 are given. From any point such as n lay off a horizontal distance $nl' = p_1$. From n lay off a vertical distance $tn = c$, and from t draw tw making the angle ϕ with the horizontal. Then draw a circle which has its center on nl', passes through l', and is tangent to tw. The distance nn' equals p_{2_a}.

To get p_{2_p}, the intensity of passive earth pressure, nn' is laid off equal to p_1, and, after a circle is drawn passing through n' and tangent to tw, the distance nl' will represent p_{2_p}.

Let c = 200 lb. per sq. ft., ϕ = 20 deg., and w = 100 lb. per cu. ft. It is desired to determine the active and passive intensities of lateral earth pressures 20 ft. below the horizontal ground surface

$$p_{2_a} = 100 \times 20 \times \overline{0.7}^2 - 2 \times 200 \times 0.7 = 700 \text{ lb. per sq. ft.,}$$

and

$$p_{2_p} = \frac{100 \times 20}{0.\overline{7}^2} + \frac{2 \times 200}{0.7} = 4{,}650 \text{ lb. per sq. ft.}$$

The total active earth pressure on the back vertical face of a 1-ft. length of wall 20 ft. high is found as follows: Owing to the cohesive properties of the soil, there will be no lateral pressure for some distance down. This distance is found by solving the right-hand side of Eq. 2-11a for h when the right-hand side is equated to zero.

$$100h \tan^2 (45 \text{ deg.} - \tfrac{20}{2}) = 2 \times 200 \tan (45 \text{ deg.} - \tfrac{20}{2})$$
$$h = 5.71 \text{ ft.}$$

A height of wall of 20 − 5.71 = 14.29 ft. is therefore subjected to a pressure varying from zero to 700 lb. per sq. ft.; hence the total pressure is 700 × 14.29/2 = 5,000 lb., acts horizontally and at a distance of 14.29/3 = 4.76 ft. above the bottom.

The passive earth pressure varies from 2 × 200/0.7 = 571 lb. per sq. ft. at the surface to 4,650 lb. per sq. ft. at the base; hence the total pressure is $(4{,}650 + 571) \times \frac{20}{2} = 52{,}200$ lb., acting horizontally and at a distance of $\frac{2 \times 571 + 4{,}650}{571 + 4{,}650} \times \frac{20}{3} = 7.4$ ft. from the bottom.

2-12. Pressure Distribution on Base of Footings. In studying the pressure distribution of a footing load to the earth beneath, we may divide our problem into three parts: (a) the distribution at the surface of contact, (b) the distribution in the disturbed zone immediately below, and (c) the distribution at points below the disturbed zone. Very little is known about the general laws of pressure distribution, either at the surface of contact or at points below. Rational formulas based on rigid footings resting on elastically isotropic masses are of but limited value since in engineering practice we are not dealing with such masses. Likewise, empirical formulas must be used with great caution since few test results are available to check their reliability. The almost infinite

variety of soils encountered makes the application of any formula highly uncertain.

Although in designing footings under symmetrical loading it is customary to assume a uniform distribution of pressure on the soil, such an assumption may be far from the truth.

For a theoretical analysis of the distribution of pressure at the surface of contact we may start with Boussinesq's classical case of the stress distribution at the base of an absolutely rigid disk, symmetrically loaded and resting on the surface of a semi-infinite elastically isotropic mass.[1] Here

$$p_z = \frac{0.5p}{\sqrt{1 - c^2}}, \tag{2-12a}$$

where p_z is the intensity of pressure at any point distant cr from the center of the disk (r being the radius of the disk) and p the average intensity of stress (the total load divided by the area of the disk). Applying this equation, we find that the intensity p_z at the center ($c = 0$) is $0.5p$ and at the edge ($c = 1$) it is infinite.

This distribution may be graphically represented by spreading the load uniformly over the convex surface of a hemisphere erected on the disk as a base. As the area of this surface is twice the area of the disk, the intensity of load on the convex surface of the hemisphere will be $0.5p$. At the center of the disk a unit area of the hemisphere projects full size on the disk; hence the intensity on the disk will be $0.5p$. At the edges of the disk a unit area of the hemisphere projects zero on the disk; hence the intensity on the disk will be infinite. At any point distant cr from the center of the disk a unit area on the hemisphere will project an area $\cos\alpha$ on the disk, where α is the angle made by a vertical with a line connecting the center of the disk with the unit area on the hemisphere. Hence the intensity on the disk is

$$p_z = \frac{0.5p}{\cos\alpha} = \frac{0.5p}{\sqrt{1 - \sin^2\alpha}} = \frac{0.5p}{\sqrt{1 - \left(\frac{cr}{r}\right)^2}} = \frac{0.5p}{\sqrt{1 - c^2}}.$$

In the case of a rigid strip having a width of $2b$ and an infinite length, the intensity of pressure at a point distant cb from the center line is given by the equation

$$p_z = \frac{2p}{\pi\sqrt{1 - c^2}}. \tag{2-12b}$$

[1] See S. Timoshenko, "Theory of Elasticity," p. 338.

In this case the graphical representation is made by distributing the load uniformly over a semicylinder instead of over a hemisphere.

In an actual footing the distribution of the soil pressure over the base may be entirely different from that expressed by the above noted equations, since no footing is absolutely rigid and few soils obey the laws of an elastically isotropic mass. The slightest deflection of the edges of the footing will tend to relieve the edge pressure; hence all edge stresses will be finite.

Krynine[1] has suggested the following empirical formula (in which n represents a concentration factor) to express the pressure distribution of an absolutely rigid round disk resting on top of an earth mass:

$$p_z = p\left(\frac{n}{2} - 1\right)(1 - c^2)^{\frac{n}{2}-2}. \tag{2-12c}$$

When $n = 3$, this equation becomes identical with Eq. 2-12a. When $n = 6$, the distribution becomes parabolic and checks the

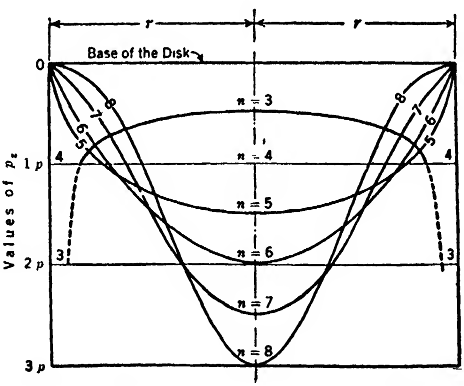


FIG. 2-12a.—Graphical Representation of Eq. 2-12c.

results of numerous loading experiments on sand masses. Figure 2-12a, taken from Krynine's paper, shows graphically the distribution of pressure on the basis of Eq. 2-12c for values of n ranging from 3 to 8.

Figure 2-12b shows the results of a test made, at the California Institute of Technology,[2] on the soil-pressure distribution over a 3-ft. square slab of concrete 10 in. thick, the total load being 23.85 tons, or 2.65 tons per sq. ft. The intensity of pressure varied from

[1] Pressures beneath a Spread Foundation, *Trans. A.S.C.E.*, vol. 103, p. 827, 1938.

[2] Converse, Frederick J., Distribution of Pressure under a Footing, *Civil Eng.*, vol. 3, p. 207, April, 1933.

a maximum of 4.2 tons per sq. ft. at the center to a minimum of 0.5 ton per sq. ft. at the corners. As shown by the iso-pressure lines, the intensity of soil pressure varied fairly uniformly from the center to the corners and from the center to the middle of the sides.

The soil was a clay-loam, made up of 29 per cent clay, 30 per cent silt, and 41 per cent sand, clay being defined as soil having particles less than 0.005 mm. in diameter, silt having particles between 0.05 and 0.005 mm. in diameter, and sand having particles between 2 and 0.05 mm. in diameter. The weight of the dry soil was 90 lb. per cu. ft. The amount of moisture at the beginning of the test was

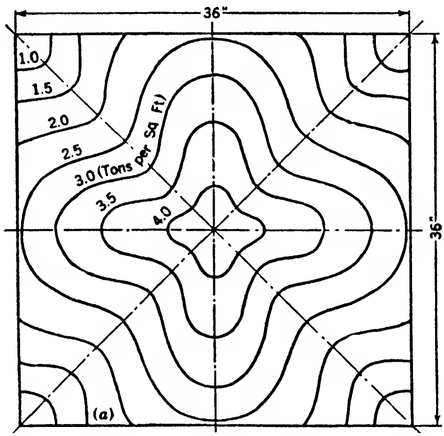


FIG. 2-12b.—Soil Pressure Distribution for an Applied Load of 2.65 Tons per Sq. Ft.

14 per cent by weight of dry material and at the end of the test 9 per cent.

The load was applied through a column composed of an 8-in. standard iron pipe having special ball-and-socket ends. The base of the column rested on steel plates, which transferred the load to the footing.

2-13. The Disturbed Zone. Immediately below a footing there is often found a zone of earth, called the "disturbed zone." In this zone bearing capacity is developed immediately on application of load by a compacting and settling of the soil to a stable condition, usually accompanied by some lateral displacement. It is in the nature of a pedestal which moves downward with the footing during its settlement. Its origin is probably quite similar to that of the cone of compressed earth which forms at the blunt tip of a pile during driving (see Art. 3-13).

Krynine[1] describes a test in which a platform 4 ft. long and 1 ft. wide, resting on fine, silty, cohesive soil, was loaded with 12 tons of pig iron. An excavation through the middle of the loaded area showed a definite semicircular disturbed zone, with the natural sand areas around and below this zone bent but not broken, as shown in Fig. 2-13*a*. The percentages noted on the illustration give the moisture content of the soil after the load was applied. It will be observed that the percentage of moisture is a minimum in the disturbed zone, this indicating pronounced soil compaction.

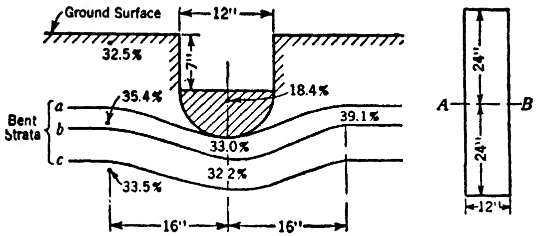


FIG. 2-13*a*.—Loading Experiment Showing Semicircular Disturbed Zone.

2-14. Pressure Distribution below Footings. In studying the stress distribution in compressible soils below a footing, we find that the distribution is in the general form of a "bulb of pressure." This bulb, which is formed by the interlocking and compacting of the soil grains, remains intact as long as the load is applied. On removal of the load it is partially dissipated. The shape of this bulb, which is fairly variable, depends on a number of factors, some of which are not well understood. Figure 2-14*a* illustrates a "bulb of pressure," as developed by Kögler and Scheidig[2] in Germany, partly from experiments and partly from theory for a 25-in. diameter bearing load resting on the surface of sand. The contours represent lines of equal vertical pressure intensities on horizontal planes, the percentages noted being in terms of the average load intensity on the contact surface of the footing.

In 1885 Boussinesq developed formulas for the stresses at any point in a homogeneous elastic solid of indefinite extent resulting from a vertical concentrated load P applied to the horizontal surface. His formula for the normal intensity of stress p_z on any horizontal plane (see Fig. 2-14*b*) is

$$p_z = \frac{3P}{2\pi} \times \frac{z^2}{R^4} \times \frac{z}{R}, \tag{2-14a}$$

[1] Pressures beneath a Spread Foundation, *Trans. A.S.C.E.*, vol. 103, p. 879. 1938.

[2] See Prentice and White, "Underpinning," p. 236.

where R is the distance from the point of application of the load to the point at which the stress is being found and z is the vertical component of R.

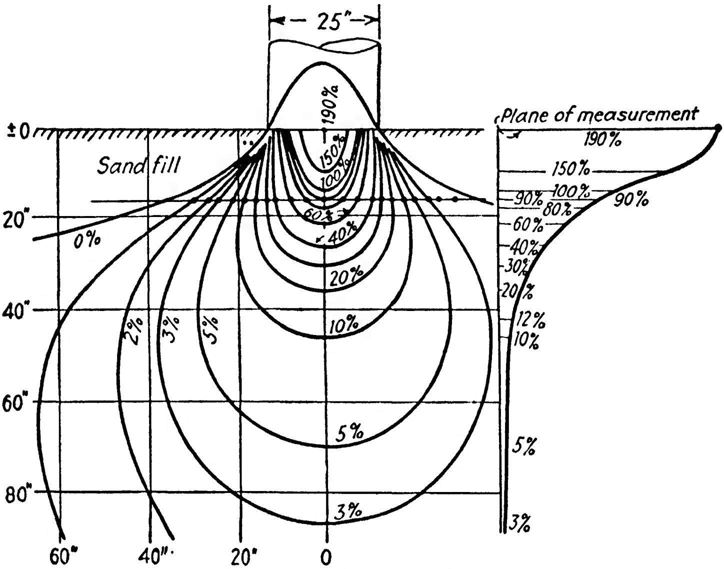


FIG. 2-14a.—Bulb of Pressure by Kögler and Scheidig.

The behavior of soils under loads is not in exact accordance with the theory of elasticity, and consequently the above formula has been modified to suit materials other than elastic isotropic solids. Griffith[1] has proposed the following formula:

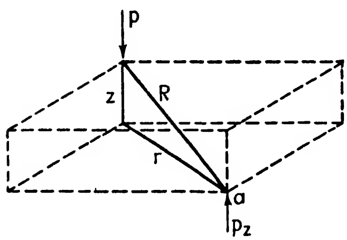


FIG. 2-14b.

$$p_z = \frac{nP}{2\pi} \times \frac{1}{R^2} \times \frac{z^n}{R^n}, \qquad (2\text{-}14b)$$

where n is a parameter that may have different values for different soil structures and loading conditions. Where $n = 3$, Eq. 2-14b is exactly the same as Eq. 2-14a.

In Eq. 2-14b, R may be expressed in terms of z and r, its vertical and horizontal projections, or $R^2 = z^2 + r^2$; hence

$$p_z = \frac{nP}{2\pi} \frac{1}{(z^2 + r^2)} \frac{z^n}{(z^2 + r^2)^{\frac{n}{2}}} = \frac{nP}{2\pi\left[1 + \left(\frac{r}{z}\right)^2\right]^{\frac{n+2}{2}} z^2}, \qquad (2\text{-}14c)$$

[1] Pressures under Substructures, *Eng. Contracting*, p. 113, March, 1929.

or

$$p_z = \frac{KP}{z^2},$$

where $K = \dfrac{n}{2\pi\left[1 + \left(\dfrac{r}{z}\right)^2\right]^{\frac{n+2}{2}}}.$

Values of K for $n = 3$ and $n = 6$ are given in Table 2-14*a* for several values of r/z.

TABLE 2-14*a*

Values of K			Values of K		
$\frac{r}{z}$	$n = 3$	$n = 6$	$\frac{r}{z}$	$n = 3$	$n = 6$
0.0	0.48	0.96	1.6	0.020	0.0060
0.2	0.43	0.82	1.8	0.013	0 0029
0.4	0.33	0.53	2.0	0.0085	0.0015
0.6	0.22	0.28	2.2	0 0058	0.00082
0.8	0.14	0.13	2.4	0.0040	0.00046
1.0	0.084	0.06	2.6	0.0029	0.00026
1.2	0.051	0.027	2.8	0 0021	0.00016
1.4	0.032	0.012			

As an example, if we assume a concentrated load of 450 tons and $n = 3$, it is required to find the soil-pressure intensity at a point $z = 40$ ft. and $r = 15$ ft. Here $r/z = \frac{15}{40} = 0.375$. Interpolating in Table 2-14*a*, we find that $K = 0.34$; hence

$$p_z = 0.34 \times \frac{450}{40^2} = 0.096 \text{ ton per sq. ft.}$$

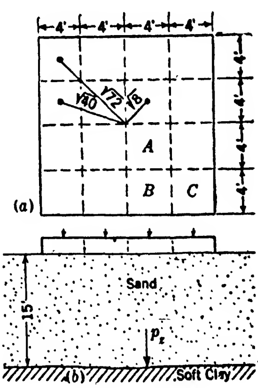


FIG. 2-14*c*.

The intensity of stress at any point due to distributed loads may be found by breaking up the distributed loads into a number of concentrated loads, the number to be selected in any case depending on the desired precision. According to Gilboy, when the area is divided into a series of equal rectangles, the longer side of which is less than one-third the distance R, the error will not exceed 3 per cent.

As an example,[1] let it be required to compute the intensity of vertical pressure on a horizontal plane at a point 15 ft. directly under the center of a raft foundation 16 ft. square (Fig. 2-14c) carrying a uniformly distributed load of 4,000 lb. per sq. ft. The raft will be subdivided into 4-ft. squares, each carrying 64,000 lb. Arranging the computations in a tabular form and using Table 2-14a for values of K, we have the following:

Square	Number of loads	Feet	$\frac{r}{z}$	K, $n = 3$	K, $n = 6$
A	4	$\sqrt{8}$	$\frac{\sqrt{8}}{15} = 0.19$	0.43	0.83
B	8	$\sqrt{40}$	$\frac{\sqrt{40}}{15} = 0.42$	0.32	0.51
C	4	$\sqrt{72}$	$\frac{\sqrt{72}}{15} = 0.57$	0.24	0.32

Using Eq. 2-14c, with $n = 3$,

$$p_z = \frac{(4 \times 0.43 + 8 \times 0.32 + 4 \times 0.24)64,000}{15^2} = 1,490 \text{ lb. per sq. ft.}$$

If the load is assumed to vary over the footing somewhat in accordance with Fig. 2-12b, the load in the A squares may be taken at 81,000 lb., in the B squares at 68,000 lb., and in the C squares at 39,000 lb. Using a value of $n = 6$, we have

$$p_z = \frac{4 \times 0.83 \times 81,000 + 8 \times 0.51 \times 68,000 + 4 \times 0.32 \times 39,000}{15^2} = 2,650 \text{ lb. per sq. ft.,}$$

or nearly twice that for a uniform distribution of load and a value of $n = 3$.

Formulas for the intensity of pressure under a number of differently shaped footings have been worked out by the integration method. Cummings[1] has worked out formulas for the pressure intensity under the center of a circular footing for a uniform distribution of load and $n = 3$, as well as for a parabolic distribution with $n = 6$.

[1] See article entitled Distribution of Stresses under a Foundation, by A. E. Cummings, *Trans. A.S.C.E.*, vol. 101, p. 1072, 1936.

Newmark[1] has developed a formula for the intensity of pressure at a point distant z directly below one corner of a rectangular area uniformly loaded (p lb. per sq. ft.) on the basis of the Boussinesq formula (Eq. 2-14a). This equation may be expressed as follows:

$$p_z = \frac{p}{4\pi}\left[\frac{2mn(m^2+n^2+1)^{\frac{1}{2}}}{(m^2+n^2+1)+m^2n^2}\,\frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\frac{2mn(m^2+n^2+1)^{\frac{1}{2}}}{(m^2+n^2+1)-m^2n^2}\right] \quad (2\text{-}14d),$$

where $m = a/z$ and $n = b/z$, a being the length of the rectangular area and b its width.

The value of the first term in the brackets ranges from zero to about 1.2, while the value of the second term ranges from zero to π.

The value of p_z at any point beneath a rectangular footing may be obtained by dividing the footing into a maximum of four rectangles, each of which has the point in question directly below the common corner.

Applying Eq. 2-14d to the problem of Fig. 2-14c, we shall have four equal rectangles (squares). Here $m = \frac{8}{15} = 0.533$ and $n = \frac{8}{15} = 0.533$ and

$$p_z = \frac{4 \times 4{,}000}{4 \times 3.14}\left[\frac{2 \times 0.533 \times 0.533(\overline{0.533}^2 + \overline{0.533}^2 + 1)^{\frac{1}{2}}}{(\overline{0.533}^2 + \overline{0.533}^2 + 1) + \overline{0.533}^2 \times \overline{0.533}^2}\,\frac{\overline{0.533}^2 + \overline{0.533}^2 + 2}{\overline{0.533}^2 + \overline{0.533}^2 + 1} + \tan^{-1}\frac{2 \times 0.533 \times 0.533(\overline{0.533}^2 + \overline{0.533}^2 + 1)^{\frac{1}{2}}}{(\overline{0.533}^2 + \overline{0.533}^2 + 1) - \overline{0.533}^2 \times \overline{0.533}^2}\right] =$$

$$\frac{4{,}000}{3.14}\left(\frac{0.712}{1.568 + 0.081}\,\frac{2.568}{1.568} + \tan^{-1}\frac{0.712}{1.568 - 0.081}\right) =$$

$$\frac{4{,}000}{3.14}(0.706 + 0.446) = 1{,}470 \text{ lb. per sq. ft.}$$

To get the intensity of pressure at a point 15 ft. below the center of one side of the area, we divide the area into two equal rectangles, for which $m = \frac{16}{15} = 1.066$ and $n = \frac{8}{15} = 0.533$. Solving by Eq. 2-14d, we find that the pressure is 1,025 lb. per sq. ft.

To get the pressure at a corner, we have a single area in which both m and n equal $\frac{16}{15} = 1.066$, in which case the pressure is found to be 730 lb. per sq. ft.

In *Circular* 24 of the University of Illinois Engineering Experiment Station, Newmark has prepared a table of coefficients for

[1] *Univ. Ill. Eng. Expt. Sta., Circ.* 24.

values of m and n varying from 0.1 to infinity. These coefficients, when multiplied by the unit load, give the pressure intensity directly, thus saving a great deal of computation work.

2-15. Settlement Studies. The 124-ft. square spread footing of the San Jacinto Monument mentioned in Art. 13-9 rests on red clay 15 ft. below the natural ground level, this clay probably extending to a depth of 220 ft. The load at the base of the footing, due to the weight at the monument, is 2.35 tons per sq. ft. In making studies to determine the probable settlement of this structure, the stress distribution was first determined on the basis of the Boussinesq formula (Eq. 2-14a), the table in the publication mentioned in the last paragraph of Art. 2-14 being used to facilitate the work. Typical stress distribution curves[1] at five points under the footing are shown in Fig. 2-15a, k being the coefficient in the equation $p_z = kp$, where p is the unit load on the footing. The points were so chosen that the average of the pressures beneath them at any elevation approximately equalled the average pressure beneath the entire footing at that elevation.

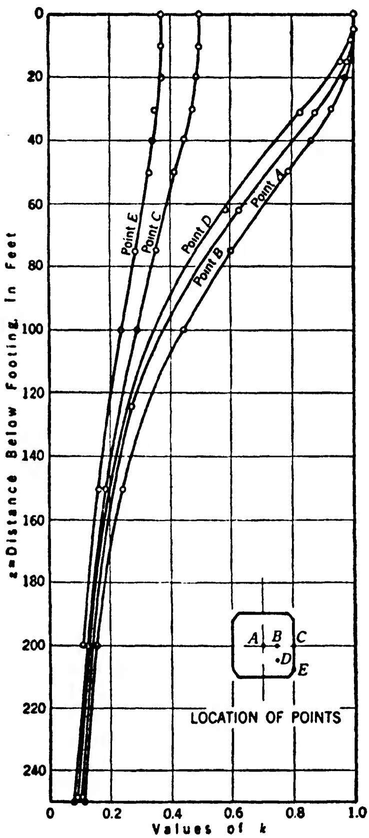


FIG. 2-15a.—Stress Distribution under San Jacinto Monument Foundation. Values of k for Use in $p_z = kp$, where p_z is the Vertical Intensity of Pressure and p Is the Unit Load on the Soil.

Samples of the undisturbed soil were obtained prior to the construction of the monument for the purpose of making consolidation tests. Figure 2-15b gives the results of a typical test, the void ratio being the ratio of the volume of voids to volume of solid material.[1] Each point on the curve indicates the void ratio after full consolidation has occurred under one increment of load.

[1] Taken from article entitled Settlement Studies on San Jacinto Monument, by Raymond F. Dawson, *Civil Eng.*, vol. 8, p. 589, September, 1938.

The probable settlement was determined by taking a number of layers and finding the settlement of each layer. The first layer of 10 ft. under the footing may be taken as an example. The initial average pressure on this layer was $(15 + \frac{10}{2})120/2{,}000 = 1.2$ tons per sq. ft., the clay weighing 120 lb. per cu. ft. The net increase in load at footing level is the difference between the applied load of 2.35 tons per sq. ft. and the weight of earth removed,

$$15 \times \frac{120}{2{,}000} = 0.9 \text{ ton per sq. ft.,}$$

or 1.45 tons per sq. ft. Reduced in accordance with the distribution curves, this became 1.13 tons per sq. ft., making the total pressure 2.33 tons per sq. ft. as an average on this 10-ft. layer.

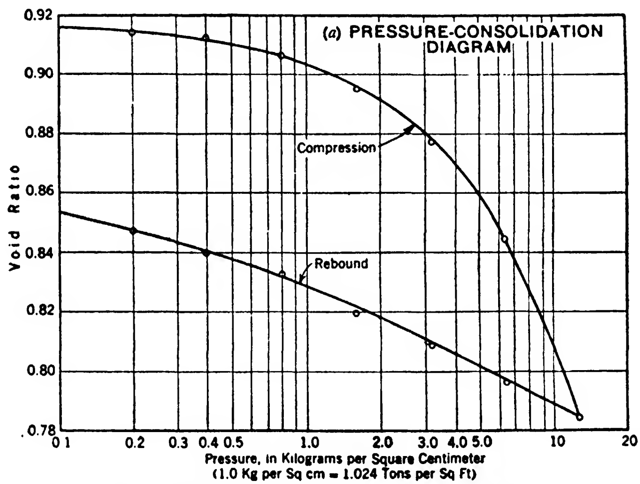


FIG. 2-15*b*.—Pressure-consolidation Diagram.

From Fig. 2-15*b* we find the initial void ratio to be 0.901 and the final void ratio 0.887. The settlement may be calculated by the formula $q = (e_i - e_f)h/(1 + e_i)$, in which q is the settlement, h the thickness of the layer, e_i the initial void ratio, and e_f the final void ratio. In our case $q = (0.901 - 0.887) \times 120/(1 + 0.901) = 0.88$ in.

In a similar manner settlements in the other layers may be found. For the entire depth the total probable settlement was found to be 7.35 in.

To estimate the rate of consolidation a time-consolidation curve can be drawn for each increment of load, the ordinates representing the percentage of consolidation and the abscissas the time. It has

been found from experience that the final 10 per cent of consolidation takes place very slowly, so the maximum observed consolidation may be taken at 90 per cent of the final value. As the rate of consolidation varies approximately as the square of the reduced thickness—thickness of the soil with all voids removed—the abscissa

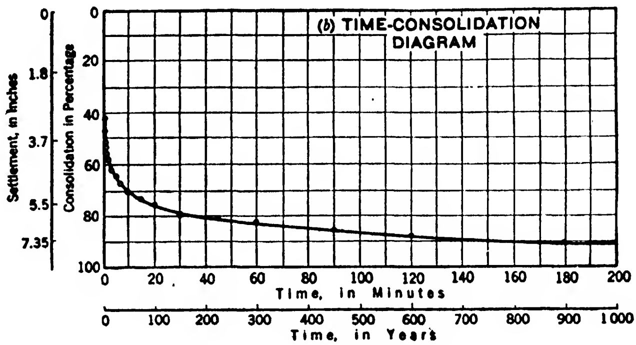


FIG. 2-15c.—Time-consolidation Diagram.

scale can be transformed from time in minutes for the sample to time in years for the actual depth. The time-consolidation curve for the footing of the San Jacinto Monument is shown in Fig. 2-15c.

The observed settlement of this monument at the end of $1\frac{1}{2}$ years was approximately $2\frac{1}{2}$ in.

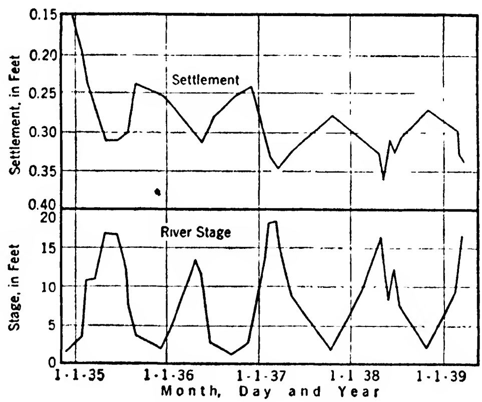


FIG. 2-15d.—Settlement of Pier 1, Correlated with River Stage.

The Huey P. Long bridge at New Orleans has foundations resting on sand at an elevation of 170 ft. below mean Gulf level (see Fig. 9-12a). Here the soil consists of alternate layers of sand and clay to an undetermined depth. The thickness of the sand layer on which pier 1 rests is approximately 100 ft., this sand layer in turn

resting on clay about 50 ft. thick. Above the sand layer are two strata of clay separated by a sand layer.

During construction and since completion of the bridge in 1935 careful measurements of pier settlements have been made. These records[1] have brought to light an interesting phenomenon, namely,

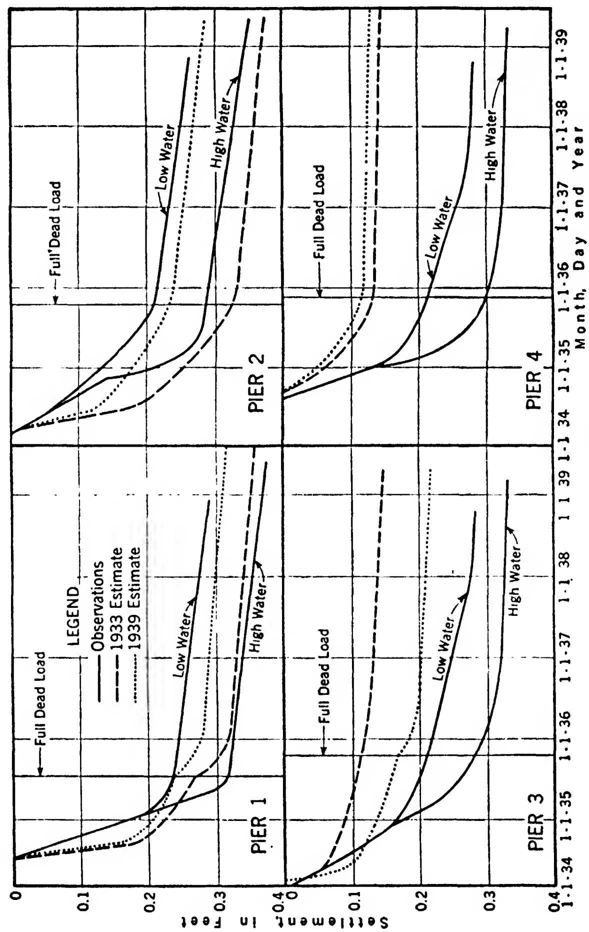


FIG. 2-15e.—Pier Settlements of the Huey P. Long Bridge. "Estimate" Curves Are for Low-water Conditions.

that the piers move downward and upward, respectively, with rise and fall of the river stage. These movements are shown in Fig. 2-15d. A similar phenomenon was observed many years ago in connection with the Pennsylvania Railroad tunnel under the Hudson River at New York City, where it was noted that the

[1] See William P. Kimball, Settlement Studies of Huey P. Long Bridge, *Civil Eng.*, vol. 3, p. 145, March, 1940.

tunnel settled 0.008 ft. coincident with a rise of tide of 4.38 ft., with an equal rise of the tunnel with a fall of tide.

The solid lines of Fig. 2-15*e* (upper left diagram) show the actual settlement of pier 1 for both low-water and high-water conditions from the time of the landing of the caisson in 1934 to 1939. It will be observed that the total settlement under low-water conditions amounts to almost 0.3 ft., of which about 0.23 ft. occurred on application of full dead load. The broken lines represent estimates based on the application of the principles of soil mechanics. The 1939 estimate is based on more complete information as to soil conditions below the plane of bearing than was available in 1933. For this pier the estimates agree reasonably well with the observed settlements. However, this is not true for all piers. For example, in the case of pier 4 (lower right diagram), the actual settlement at low water was approximately 0.28 ft., when the estimated value was only 0.15 ft.

Quoting Mr. Kimball:[1]

Consolidation of the clay strata accounts for approximately 50 per cent of the observed settlement, the remainder apparently taking place in the sand. In preparing the settlement estimates it was assumed that pier settlement caused by these deep-lying sand strata could be approximated from the results of consolidation tests on the undisturbed sand samples. The estimates have been criticized for this assumption. It is believed that the settlement of structures on sand is caused jointly by vertical compaction and by lateral yield. The former can be estimated by the widely accepted theory of consolidation, whereas no reliable method of forecasting the settlement caused by lateral yield has yet been developed. . . . The settlement of a loaded plate on the surface of a bed of sand is believed to be caused almost entirely by lateral yield, the vertical compaction being a relatively negligible factor. The writer submits, however, that the settlement of a loaded plate buried to a great depth is caused almost entirely by vertical compaction, the lateral yield, owing to confinement of the overlying material, being a relatively negligible factor. The piers of the New Orleans bridge are believed to lie somewhere between these two extremes. . . .

Another important source of uncertainty, particularly in cases of deep piers subject to only small settlements, is the condition of the bearing stratum on which the structure rests. Undoubtedly a considerable early settlement may be due to the surface adjustment of soil which has been disturbed during the preparation of the bottom. Unfortunately the amount of such settlement will always be unpredictable.

[1] See William P. Kimball, Settlement Studies of Huey P. Long Bridge, *Civil Eng.*, vol. 3, p. 145, March, 1940.

It has been further assumed that all the load is transmitted to the soil at the bottom of the caisson seal. The load that might be permanently transmitted through skin friction along the sides of the pier has never been determined.

In spite of extensive research on the distribution of stress through soil, experimental verification of current stress theories is meager and conflicting; there is no widespread agreement on methods of calculating stress; and the theories that have been advanced lead to widely varying results. In the settlement analysis of these piers made in 1933, stresses were computed on the basis of Boussinesq's classic equation. The determination of stresses at various locations and depths, and the weighing of these to determine the average stress at ruling elevations under each pier, was a laborious process requiring approximately 100 man-hr. The average stresses at the same elevations under each pier were recomputed in the preparation of this paper in less than 2 hr. on the assumption that the load on the pier base is spread through the underlying soil between planes making an angle of 60 deg. with the horizontal.[1] . . . Applying the stresses computed by this simplified method to this particular settlement analysis results in differences in the estimated ultimate settlements of less than 5 per cent.

2-16. Theory of Bearing Capacity. For many years it has been general practice to adopt unit bearing values based on soil type only. That this is not sound practice has been amply demonstrated

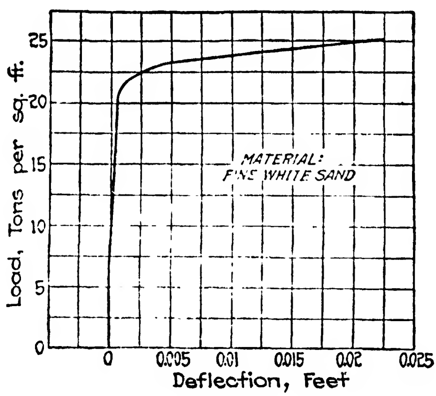


FIG. 2-16*a*.—Typical Load Settlement Diagram for Sand Foundations.

in recent years. The amount of settlement to be expected for any given unit loading depends on many factors besides soil type; among these are the degree of soil consolidation, the size of the bearing area, and the depth of bearing area below the surface.

As explained in Art. 2-2 settlement results from soil consolidation and from lateral displacement of the soil. Consolidation is due to

[1] See Art. 1-15.

compressive stresses, while lateral displacement is caused by shearing stresses. Soil consolidation is directly proportional to the load intensity, but lateral displacement may be quite independent of load intensity. There is considerable evidence indicating that settlement due to shearing stresses is approximately proportional to the perimeter of the footing. In most loading tests, settlements first increase proportionately to the load applied, but, when the load reaches a certain critical value, the settlement increases disproportionately (Fig. 2-16*a*). The sudden break in the curve probably represents the point at which shearing failure occurs. According to Terzaghi,[1] for cohesive soils the settlement produced by a given unit load increases in direct proportion with the diameter of the loaded area, whereas for cohesionless soils settlement produced by a given unit load is practically independent of the size of the area.

Load tests are usually made on areas that are very small compared with the footing sizes (Art. 1-16). In the light of the above discussion it is evident that the results of load tests made on cohesive soils must be applied with caution to large bearing areas. The effect of a load on a small area is not reflected through as great a depth as a load on a large area (Art. 2-14); hence, where a test is made on a sand or gravel overlying soft clay, the pressure on the soft clay due to the test load may be negligible, whereas the pressure on this stratum from the finished structure may be considerable.

W. S. Housel has suggested the use of the following equation for the design of footings, all forces being in pounds and all distances in feet:

$$W = mP + nA, \tag{2-16a}$$

where W = total load in bearing area
P = perimeter of footing
A = bearing area of footing
n = developed pressure under bearing area
m = developed perimeter shear or boundary reaction

This equation may be rewritten as follows:

$$w = \frac{W}{A} = m\frac{P}{A} + n. \tag{2-16b}$$

The boundary effect $\left(m\frac{P}{A}\right)$ is thus expressed as an equivalent uniform pressure, which, added to the developed direct pressure n, will give the total equivalent unit load that can be placed on the bearing area without exceeding a certain permissible settlement.

[1] The Science of Foundations, *Trans. A.S.C.E.*, vol. 93, p. 270, 1929.

The values of m and n may be determined experimentally by running load tests on at least two different size bearing plates and placing the simultaneous values of W and A at the permissible settlement in Eq. 2-16*b*.

For uniform soil conditions to an indefinite depth, the intensity of load that can be carried with any permissible settlement will increase with the depth of the bearing plane below the surface. This is due to (*a*) a higher degree of soil consolidation and (*b*) a higher resistance to lateral displacement because of the adjacent overburden. However, according to Terzaghi, the permissible load does not depend on the depth alone but on the ratio t/d, where t is the depth and d the diameter of the bearing area. He also states that the influence of this ratio is greater for cohesionless soils than for cohesive soils.

CHAPTER III

TIMBER PILES AND DRIVERS

3-1. Classification of Piles. A pile is an element of construction placed in the ground, either vertically or nearly so, to increase its power to sustain the weight of a structure or to resist a lateral load. A pile is usually placed in position in the ground by driving it with a steam-hammer or with a drop-hammer, either with or without the aid of water-jets. In rare instances a pile may be sunk into place by static pressure. Sand piles and certain types of concrete piles are, however, cast directly in place. The principal use of piles occurs in the foundations of bridges, buildings, and other structures, in which they act as bearing piles.

Piles are designated by the material of which they are composed, as timber piles, concrete piles, metal piles, and sand piles; by their use, as bearing piles, batter piles, guide piles, fender piles, and sheet piles; or by some attachment to their feet in order to increase their bearing power, as lagged piles, disk piles, screw piles, or bulb piles.

The materials employed for piles include wood (treated or untreated), concrete (plain or reinforced), cast iron, wrought iron, steel, and sand. Sometimes two materials are used in combination, as, for example, in a timber pile surrounded by a protection of reinforced concrete, or in a hollow metal pile filled with concrete. Piles composed of sand are made in place in the ground in a vertical cavity formed for that purpose and hence serve chiefly to compact the earth and thereby increase its bearing capacity.

A bearing pile is one that carries a superimposed load. Its form of cross section depends on the material of which it is composed, and it may be round, square, octagonal, annular, or H-shape. Its longitudinal section is frequently tapering, but sometimes its cross section remains constant throughout its length.

Bearing piles are used in foundation construction under the two following typical conditions: (*a*) when the piles are driven through soft material to or into a stratum of firm or practically unyielding material, and (*b*) when no hard bottom can be reached with any reasonable length of pile and the friction of the earth on the pile is sufficient to support the load with safety. In the first case the

pile receives little, if any, lateral support and therefore acts as a column. In the second case true pile action develops and the load is limited either by the friction of the earth or by the compressive strength of the upper part of the pile.

The most favorable condition for the use of bearing piles occurs when a firm stratum can be reached by piles of ordinary dimensions (hence easily obtainable in the markets) and the overlying material is compressible (hence easily penetrated by piles) but sufficiently compact to prevent the piles from bending and being displaced laterally.

The head of a pile is its upper end; the foot of a pile is its lower end; the butt of a pile is its larger end; and the tip of a pile is its smaller end. These definitions show that the terms "head" and "foot" relate to the pile in its final position only, while the terms "butt" and "tip" apply to a tapered pile either before or after it is placed in position.

A batter pile is one that is driven at an inclination to resist forces that are not vertical. They are sometimes called "spur piles." When a pile structure is built to resist lateral forces, experience has proved the importance of relying chiefly on batter piles, rather than on cross bracing of vertical piles, to ensure lateral stability.

Guide piles are used principally in cofferdam construction to support the horizontal timbers or wales; which in turn, support vertical sheet piling (Art. 8-3). They are also used in ferry slips and to aid in locating and sinking open and pneumatic caissons.

Fender piles, as their name implies, are driven at wharves or in front of large masonry structures or other important works, to protect them from sudden blows by ships.

Sheet piles are used to resist lateral pressure of earth and to form a wall which is intended to be watertight. Their form usually differs from that of other piles, there being a considerable variety of types of cross sections (Chap. VII).

3-2. Timber Piles. The desirable qualities of timber piles relate chiefly to strength and durability. In the specifications (1936) of the American Railway Engineering Association, wood piles are divided into two classes, first and second. The species included in first-class piles are cedars, chestnut, cypress, Douglas fir, larch, oaks, pines, spruces, and tamarack. Second-class piles, used for foundations that will always be submerged and for cofferdams, falsework, and other temporary work, may be of any sound timber that will stand driving.

These specifications provide that first-class piles shall be free from any defects which may impair their strength or durability as piling, such as decay, splits, twist of grain exceeding one-half of the circumference in any 20 ft. of length, or shakes more than one-third of the average diameter of the pile. The maximum diameter of a knot shall not exceed 4 in. nor one-third the least diameter of the pile section where it occurs. Knots in clusters or groups shall not be permitted. Holes shall not exceed $1\frac{1}{2}$ in. in diameter nor be deeper than one-fifth of the diameter of the pile where they occur. If close grain is specified for softwood piles, they shall show on the butt end not less than six annular rings per inch, measured radially over the outer 3 in. of the cross section. Douglas fir and southern pine averaging five to six annular rings per inch shall be accepted as close grain if they have one-third or more summerwood. Piles for use without preservative treatment shall have as little sapwood as possible; piles for use with preservative treatment shall have as much sapwood as possible—in southern pine a sap ring of not less than $1\frac{1}{2}$ in. and in Douglas fir of not less than 1 in. at the butt end.

For second-class piles the specifications are the same as given in the first sentence of the preceding paragraph. The principal differences in the requirements of the two grades are with reference to strength and durability.

Southern pine, Douglas fir, and white oak are among the most valuable species of wood used in modern engineering construction. Cedar piles are noted for their long life and durability. Beech, ash, and basswood are used to a limited extent for piles. In Florida, palmetto piles are used, as this wood is somewhat resistant to marine-borer attack. White-pine piles were used in the northern central states before the close of the nineteenth century, but since then this species has become too valuable on account of its demand for other uses in building construction. Southern pine, Douglas fir, spruce, cedar, and other conifers have increased values for piles because they are straight and free from large branches. The longest piles used in single sticks are Douglas fir. Oak piles are hard and tough but are not so straight and smooth as the conifers and have the further disadvantage, on account of weight, of increased cost of transportation and of liability to sink in water unless lighter logs are used in rafts to buoy them up.

The specifications noted in the first paragraph of this article require that piles shall taper uniformly from the point of butt measurement to the tip, that they shall be free from short or

reversed bends, and free from crooks greater than one-half the diameter of the pile at the middle of the bend. A line drawn from the center of the butt to the center of the tip shall lie within the body of the pile. All knots and limbs shall be trimmed or smoothly cut flush with the surface of the pile. First-class piles shall be peeled smooth, clean, and free of inner bark.

The standard specifications (1937) of the American Society for Testing Materials for timber piles provide for three classes, *A*, *B*, and *C*; class *A* being for heavy railway bridges and trestles, class *B* for docks, wharves, highway work, and general construction, and class *C* for foundations that will be completely submerged, cofferdams, falsework, and sundry temporary work.

3-3. Durability of Timber Piles. Foundation piles, when cut off below the ground-water level, apparently have an indefinite life. For example, in reconstructing a bridge, timber piles were removed which indicated no material decay after being in service 600 years. A still more conspicuous example was brought to the attention of engineers and architects when the campanile of St. Mark's in Venice fell in 1902. The piles in the foundation, which had been in service for 1,002 years, were found to be in such a good state of preservation that they were allowed to remain to support the reconstructed tower. However, where piles extend above ground-water level, they must be impregnated with creosote or some other germicide.

The time of the year in which timber is cut for piles may greatly affect both its durability and its strength. Tests made in Germany of four spruce trees, growing close together in the same soil, showed that if the strength, when cut in December, is taken as 100 per cent, those cut in January, February, and March had strengths of 88, 80, and 62 per cent, respectively. Beech timber cut in December and January gave an average mechanical life of 6 years, whereas the same kind of timber cut in the same location on February and March gave a service of only 2 years.

Experience in this country has also shown conclusively that the use of piles of the best species of wood may lead to serious loss when it is cut in the summer and left only a short time before the bark is peeled. Decay due to fungi and the ravages of worms, which become manifest when the sapwood began to decay, required, in one case involving a very large number of piles, the replacement of the whole lot within 4 years, some of them being eaten through entirely within 2 years.

3-4. Form and Dimensions. Since a timber pile generally consists of the lower portion of the trunk of a tree, after its branches

and bark are removed and the knots trimmed close to the body, its cross section is approximately circular. Square piles are rarely used as bearing piles and only to a limited extent for special purposes, one of which is to form large timber sheet piles by the addition of scantlings on two sides to form tongue-and-groove joints.

The specifications referred to in the first paragraph of the preceding article also provide for limiting sectional dimensions, namely, the permissible minimum diameter at the tip and the permissible minimum and maximum diameters 3 ft. from the butt. The values vary with the class of piles, the species of wood, the

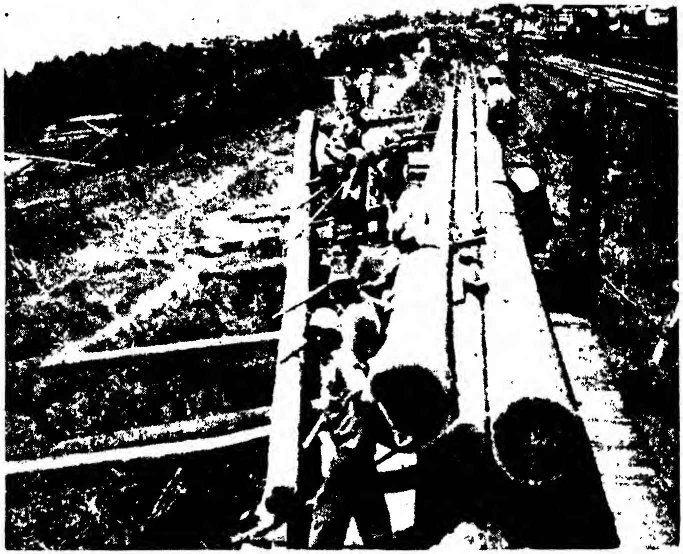


FIG. 3-4a.—Piles 135 Ft. Long Used on Falsework of Baton Rouge Bridge.

length of the pile, and the use (whether for railway or highway bridges). For example, southern pine and Douglas fir piles under 40 ft. in length used for railway bridges must have a tip diameter of not less than 10 in. and a diameter at a point 3 ft. from the butt not less than 14 in. or more than 18 in. For longer piles the minimum tip diameter is reduced and the butt diameter increased. For example, sticks of southern pine or Douglas fir over 90 ft. long used for highway bridges may have a tip diameter of 5 in. and a diameter 3 ft. from the butt of 20 in.

The required length of a pile necessarily depends on the character of the earth into which it is driven. Piles as short as 10 ft. have been used, but it is questionable whether this is not too low a

minimum. In ordinary construction the length of piles varies roughly from 20 to 40 ft. As an illustration of the use of long timber piles, in the jetty construction at the mouth of the Columbia River, piles in single sticks 130 ft. long and 30 in. in butt diameter were driven 50 ft. into the bed of the river. Other examples include unspliced piles 135 ft. long (Fig. 3-4*a*) used at Baton Rouge in falsework construction, and single sticks 129 ft. long used in building a terminal pier at Portland, Maine. Single lengths 126 ft. long and spliced lengths 150 ft. long were used for the foundations of a building in San Francisco (see Fig. 3-6*b* for driver used). Even greater lengths up to 175 ft. were used on the Pacific Coast, but, since the best timber near the coast or navigable streams has been cut, the available lengths are limited by the conditions of railroad transportation. Where the character of the earth or of the several strata to be penetrated is fairly uniform over the area of the given site, it is desirable to use piles as nearly alike in diameter and length as can be secured economically in the available markets.

The principles of good design and economic construction require that the proper lengths of piles be determined in advance. In the absence of definite knowledge by previous pile-driving experience at the same location or contiguous to it, a careful exploration of the ground should be made by means of auger or wash borings or by means of test piles. Tests should be made at certain intervals along the line of a trestle bridge, at the locations of piers and abutments, or at several places distributed over the area of a building foundation. Driving test piles is advantageous, since it furnishes information at the same time on the number of blows required to secure the necessary total penetration and hence the approximate time for the subsequent work.

3-5. The Phenomena of Pile Driving. The term "pile driving" is applied to the operation of taking a pile and forcing it into a definite position in the ground without previous excavation. A number of methods are employed for this purpose which require different kinds of equipment. Historically the oldest method of driving a pile is by means of a hammer. While very small bearing piles, or posts, were doubtless driven at first by hand with a maul or beetle, those of larger size, usually designated as piles, required the use of a machine by which a hammer was raised with the aid of a pulley and rope and allowed to drop on the head of the pile. A weight used in this manner was hence called a "drop-hammer." At first men, then horses, and afterward the steam engine were used to raise the hammer.

After the invention of the steam engine, steam-hammers were designed in which the driving weight is lifted a short distance by steam pressure and allowed to fall by gravity, the rapidity of action being greatly increased. Subsequently steam-hammers were invented in which steam pressure reinforces the action of gravity on the downstroke. Most pile driving is now done by the use of steam-hammers, the drop-hammer being used only for small jobs where it is uneconomical to install the heavier and more expensive equipment required by steam-hammer operations. At one time pressure due to the explosion of gunpowder was used to drive piles, but that method is now regarded as antiquated. To a very limited extent pile driving has been accomplished by placing a static weight upon a pile and rocking it to and fro in soft ground, to which condition this method is practically limited.

Another method of more recent discovery, which has greatly advanced the art of pile driving, consists in the use of the water-jet to aid in displacing the earth at the foot of the pile and to lessen the friction of the pile as it descends through the surrounding material. This method is generally employed in conjunction with the use of a hammer, although occasionally the hammer may serve merely as a static weight during a portion of the time required to sink the pile. The latest development is that of preboring the holes and then driving the piling for the final few feet of penetration.

The phenomena of pile driving may perhaps be most readily understood by the student by considering the case in which a timber pile is driven vertically into the ground by means of a drop-hammer. After the piles are delivered on the site within reach of one of the lines of the pile driver which is used to handle the piles, the line is made fast to a pile near its head, is first dragged, if necessary, close to the front of the pile driver, and then hoisted until it is suspended in the air. It is next placed and held laterally between the pair of tall parallel members of the pile driver, known as the "leads," between which the hammer is guided in its movements. After lowering the pile until its foot rests on the ground, the line is released. The hammer, being held at the top of the leads by the other line, is now released and in falling strikes the head of the pile. It is then raised again and released for the second blow, and so on for successive blows until the required penetration of the pile is obtained.

During its fall, the velocity of the hammer is increased until it reaches a maximum at the instant of striking the pile. The energy of the hammer at this instant equals the product of its weight and

the free fall, minus losses due to friction and other effects. On striking the pile there are certain energy transformations in conformity with the laws of impact (Art. 4-1). Some of the energy remains in the hammer, some is lost in heating and brooming the head of the pile, some is dissipated in elastic compression of the pile and surrounding earth, and the remainder causes the penetration of the pile.

The resistance to penetration is caused by a combination of point bearing, the axial component of side pressure, and side friction. Small-sized experiments on pressing sticks with blunt tips into sand and other kinds of earth, as well as observations of regular piles, show that a compact cone of earth is formed under the blunt tip and remains there, being pushed along through the ground as the stick descends. The upward force from this cone, together with the vertical component of the normal earth pressure against the sides of the pile, where the same are driven tip down, form one element of driving resistance. It is thus seen that the resistance of the soil to displacement is one of the elements determining the load which a pile can bear. In most cases this resistance increases more or less with the depth.

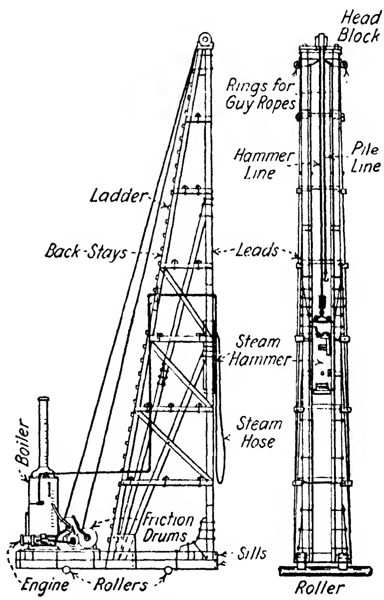


FIG. 3-6a.—Typical Timber Skid Pile Driver.

The displaced soil in contact with the pile also develops a resistance to pile penetration by friction, the total amount equaling the normal force times the coefficient of friction between the pile and earth. If this coefficient is larger than that between earth and earth, then the latter will govern, a thin layer of the earth adhering to the pile. As the load is taken from the pile by the earth, it is distributed out over horizontal areas at lower elevations in the form of a conoid of pressure, the slope of which depends on the physical characteristics of the soil, a commonly used figure being 60 deg. with the horizontal.

3-6. Pile Drivers. Two general classes of outfits are used for placing and driving piles, (*a*) those designed solely for pile driving

and (*b*) those designed for other work in addition to pile driving. The first class may again be subdivided into three groups, namely, track pile drivers (Fig. 3-6*c*), skid pile drivers (Fig. 6-7*c*), and floating pile drivers (Fig. 7-4*a*). Most track pile drivers can travel under their own power, and the table of the driver can be rotated by means of the driving machinery.

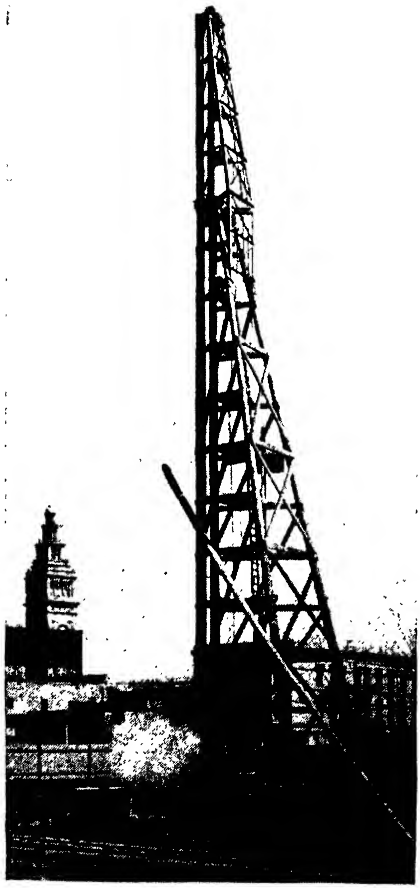


FIG. 3-6*b*.—Timber Pile Driver Surmounted with a Steel Frame Extension.

Figure 3-6*a* illustrates the standard timber skid pile driver assembly as used 25 years ago. Its characteristic feature consists of the leads, which are upright parallel members supporting the sheaves used to hoist the hammer and piles. They also guide the hammer in its movement, the inner faces of wooden leads being lined with channel irons in order to reduce friction and wear. The

leads are held in position by being framed with backstays and other bracing into the form of a tower supported on horizontal sills. When used as floating equipment, the pile driver is mounted on a barge. When used on land, the bedframe containing the sills is extended back far enough to support the hoisting engine and boiler and the whole is mounted on rollers. The tower may be braced

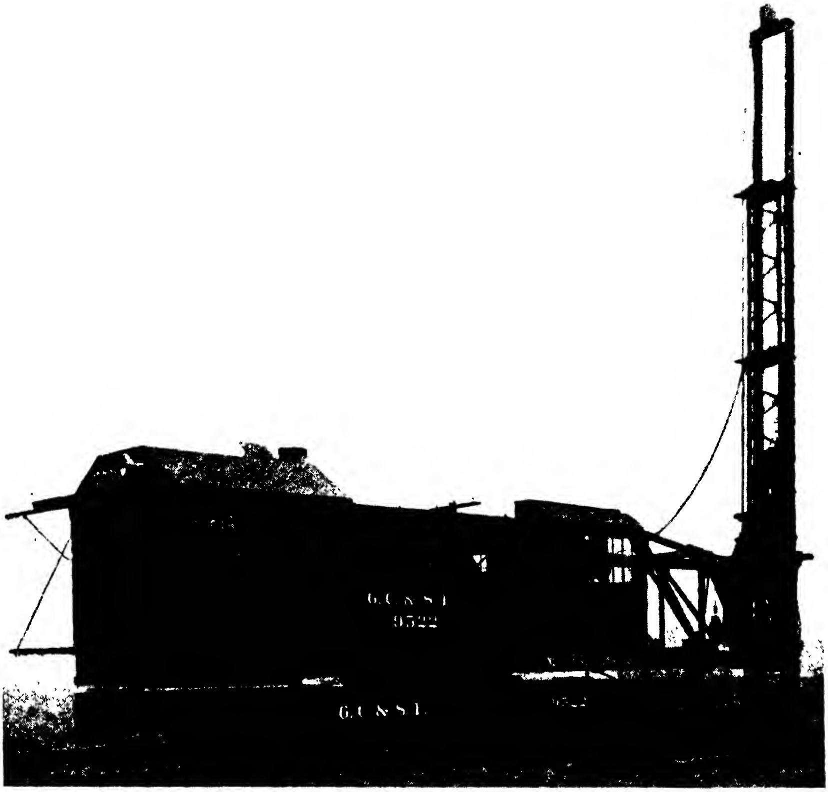


FIG. 3-6c.—A Swiveling Track Pile Driver. (*Courtesy of Industrial Brownhoist Corp.*)

either by guy lines attached to rungs near its top or by long inclined posts, or wind braces, in which case the bedframe is generally widened to support these braces. Leads as long as 100 ft. under the head block have been used. Figure 3-6*b* illustrates a timber pile driver 100 ft. high surmounted with a steel-frame extension 35 ft. high, constructed to drive spliced piles 150 ft. long.

By the use of two sets of rollers at right angles to each other, a driver may be moved forward, backward, or sidewise. When it is mounted on a turntable, it is called a "swiveling" pile driver and

it combines swinging to the right or left with the motions noted in the preceding sentence.

Telescopic or extension leads which slide inside of the stationary leads are commonly used to drive piles below the elevation of the pile driver. By this means piles may be driven without the use of a

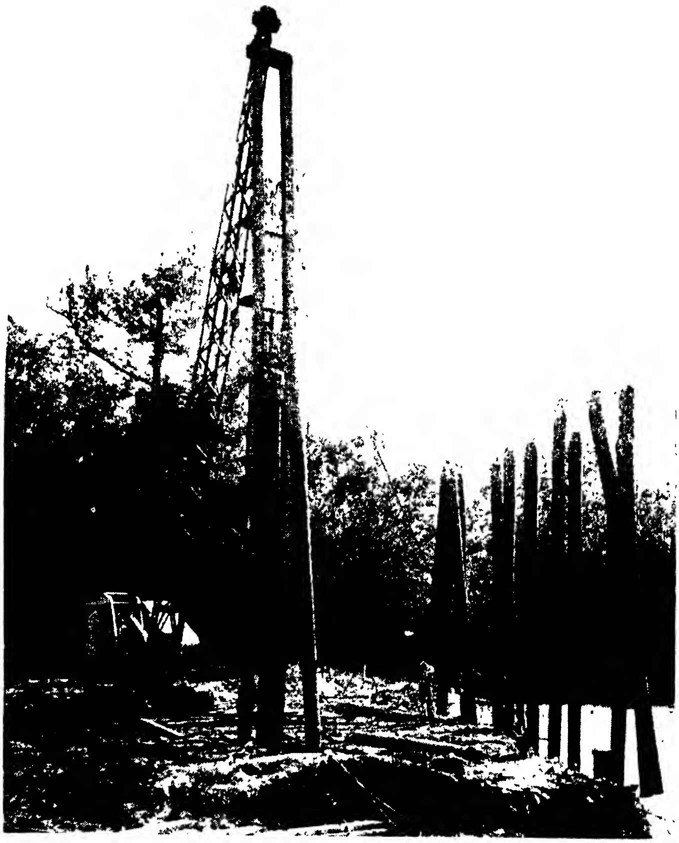


FIG. 3-6*d*.—Caterpillar Tractor Crane Assembly Equipped with Timber Hanging Leads.

follower in deep trenches, or through contracted openings, or in the bottom of cofferdams containing a large amount of internal bracing. Floating pile drivers have had, in exceptional cases, 100-ft. telescopic leads working within 100-ft. fixed leads.

Pile drivers may be built of timber or of steel. When built of timber they generally conform to the description given above. Steel pile drivers are built in a variety of forms for different pur-

poses or conditions. Figure 3-6*c* illustrates a typical swiveling track pile driver. When not in use, the leads are lowered by rotation to rest on top of the car. Figure 7-4*a* illustrates an up-to-date floating steel pile driver used for driving long steel H-beam piles, in which the total weight of hammer, including anvil blocks, was 31,800 lb.

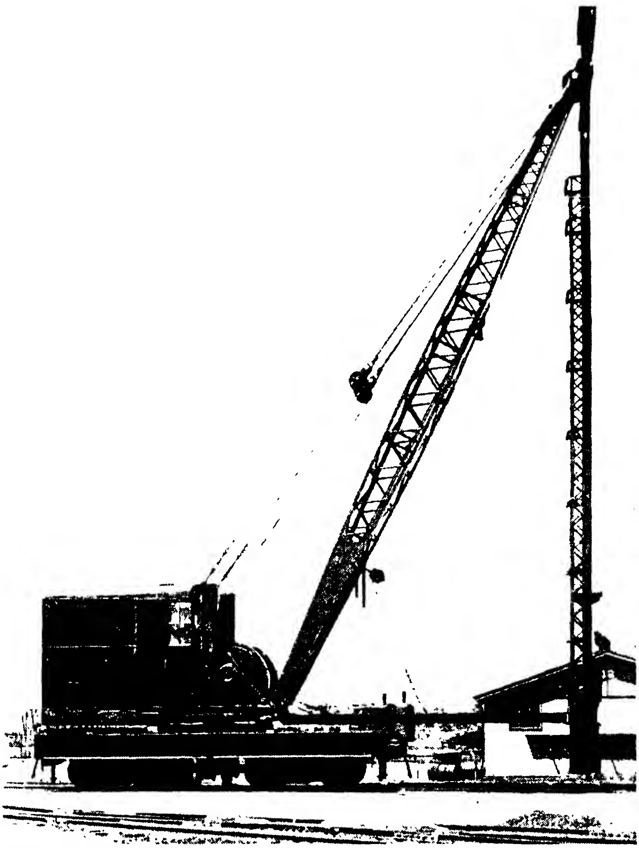


FIG. 3-6*c*.—Diesel Locomotive Crane Equipped with Hanging Steel Leads.

Figure 6-7*c* illustrates a modern steel skid pile driver as used by the Raymond Concrete Pile Company in driving the shells for their type of pile (Art. 6-7).

Except in the case of large jobs where the equipment is specially designed, the present tendency, particularly for driving timber piles, is toward the use of cranes and derricks mounted on crawlers, cars, or scows. Such equipment can be used for many purposes in the construction field in addition to that of pile driving. Generally, but

not always, these cranes and derricks are equipped with demountable hanging leads, which also serve as extension leads. Figure 3-6*d* shows a caterpillar tractor crane assembly equipped with timber hanging leads, while Fig. 3-6*e* shows a modern Diesel locomotive crane equipped with hanging steel leads.

Figure 7-8*a* shows pipe piles (Art. 7-8) being driven without the use of leads, the hammer in the foreground being suspended from the derrick with a line just taut enough to hold the hammer in place. The hammer in the background has no support other than the cap in which it rests atop the pile.

3-7. Drop Pile-hammers. A drop-hammer is one which is raised by means of a rope and then allowed to drop. It consists of a solid casting with jaws on each side which fit into the guides of the pile driver leads, with a pin near the top for the attachment of the rope or of the nippers, and with a broad base on which it strikes the pile.

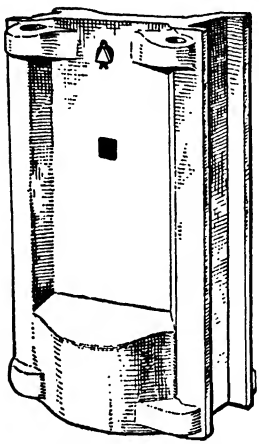


FIG. 3-7*a*.—Drop-hammer.

Figure 3-7*a* shows a drop-hammer of modern design with all corners rounded. It is made as long as practicable to increase the bearing in the leads, while the jaws have as little play as possible between the leads and hammer to reduce the jar on the driver when the pile is struck. The form is arranged to have its center of gravity as low as possible. When the hammer is to hit the head of the pile directly, its base is made slightly concave, but, when a pile cap is employed, as is done in the best practice, the base is made flat.

When the hammer is to have a free fall, as may be required on test piles, the pin is triangular in section with its lower face horizontal, to engage the "nippers" automatically. The upper ends of the nippers are curved so that, when the trip is reached, they are drawn together and thus release the hammer for its drop on the pile. When the hammer is to be raised by a hoisting drum with a friction clutch, a round pin is used to which the line is attached directly. The latter method affords the following advantages for regular work: more rapid operation, facility in regulating the height of drop, and avoiding the danger of losing the hammer if it should pass out of the leads.

The weight of drop-hammers most generally used in American practice to drive timber piles ranges from about 2,000 to 3,800 lb.

For posts and very small piles the weight runs as low as 500 lb.; for heavy construction requiring very long piles, it runs as high as 5,200 lb. For very light service a heavy block of oak wood is sometimes employed. The weight of drop-hammers to be adopted depends upon the weight of the piles and the character of the ground to be penetrated. The weight of hammers to drive timber and concrete piles is referred to in Arts. 4-3 and 6-15, respectively.

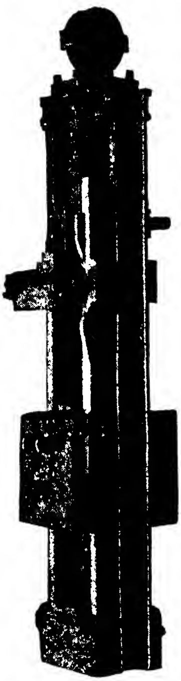


Fig. 3-8a.—Warrington-Vulcan Steam Pile-hammer.

Fig. 3-8b.—Industrial Brownhoist Steam Pile-hammer.

3-8. Steam Pile-hammers. A steam pile-hammer is one that is automatically raised and dropped a comparatively short distance by the action of a steam-cylinder and piston supported in a frame which follows the pile. Invented in England by James Nasmyth, it was first used in 1845 in driving piles for the Devonport Dock of the Royal Navy. Its first application to the driving of piles for a bridge foundation was on Oct. 6, 1846. One type of steam-hammer has been built in this country since 1875 and, after various

improvements, has continued in use, being known at present as the Warrington-Vulcan hammer. Steam-hammers are of two general classes, single acting and double acting. In the first and older class the steam pressure is applied to raise the striking part of the hammer, while it falls by gravity. The force of the blow depends on the length of stroke and the movable weight, the number of blows per minute depending on the steam pressure. In the second class the steam pressure raises the hammer and also reinforces the

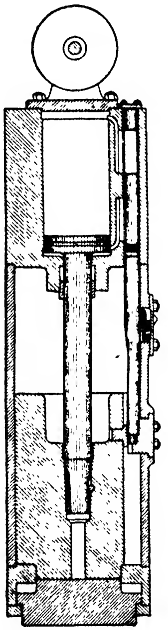


FIG. 3-8c.—McKiernan-Terry Steam Pile-hammer.

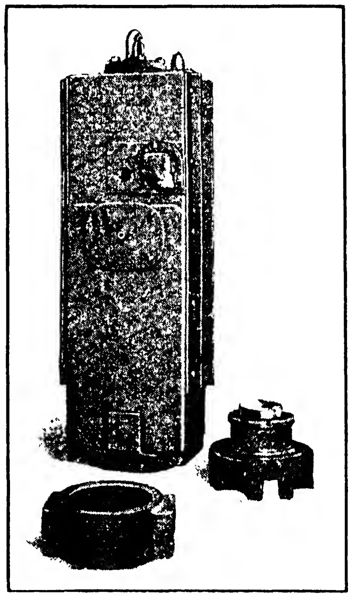


FIG. 3-8d.—Union Steam Pile-hammer.

action of gravity during its descent, the force of the blow, as well as the rapidity of action, being functions of the pressure. Double-acting hammers are more compact, are lighter, and operate with greater rapidity.

The Warrington-Vulcan hammer (Fig. 3-8a) is single acting, while the Union (Fig. 3-8d), Industrial Brownhoist (Fig. 3-8b), and McKiernan-Terry (Fig. 3-8c) hammers are double acting. The Super-Vulcan resembles the double-acting hammers in that steam pressure reinforces the action of gravity during the descent of the hammer but differs from them in that the steam below the piston is never exhausted; its pressure is constant. It resembles the single-acting Warrington-Vulcan in having a ram that is heavy with

respect to the total weight of the hammer, but differs from the same in having a much shorter stroke and higher frequency of blows.

Another classification may be based on whether the striking part is attached to a moving piston or to a moving cylinder. The Warrington-Vulcan, Union, and McKiernan-Terry hammers have the first-named arrangement; the Industrial Brownhoist has the second.

The following table gives weights, dimensions, and other data for the largest regular size of hammer for each of five makes. Most steam-hammers may also be operated by compressed air.

Largest Sizes of Various Steam Pile-hammers

Trade designation	Warrington-Vulcan	Union	Industrial Brownhoist	McKiernan-Terry	Super-Vulcan
Size number	0	00	.....	11-B-3	20,000
Total weight, pounds	16,250	21,000	6,600	14,000	39,050
Weight of striking part, pounds	7,500	6,000	1,900	5,000	20,000
Total height, inches	180	......	113	133½	154
Diameter of cylinder, inches	16.5	14	8	11	
Stroke, inches	39	36	24	19	15½
Work done per blow, foot-pounds	21,375	54,900	9,659	19,150	50,200
Number of blows per minute	50	85	110	95	98
Steam pressure, pounds per square inch	80	100	90	....	142
Boiler required, horsepower	60	125	40	60	120

The total weights of the smallest sizes are, respectively, 1,400, 100, 6,000, 105, and 1,832 lb. The number of blows per minute for the same sizes are 80, 550, 150, 1,000, and 225. Generally, the lightest hammers are used for light sheet piling only. Additional data may be found in the illustrated catalogues published by the manufacturers.

Some manufacturers make hammers of special designs. For example, the McKiernan-Terry Corporation have made a double-acting hammer weighing without the anvil block 28,000 lb., or including the anvil block, 33,000 lb. The ram weighs 10,000 lb. and delivers 55,000-lb. of energy per blow. The same company also made a single-acting hammer having a total weight of 31,800 lb., including anvil, and a ram weight of 14,000 lb.

During the operation of driving, the steam-hammer and its frame rest upon the pile, the head of which is trimmed to fit into the

recessed or open base of the frame. The frame has channel or angle guides on the sides which engage the leads of the driver. The frame in turn guides the hammer in its movement, and in several makes entirely encases it. While the weight of the striking parts is only a fraction of the total weight, the extra dead weight of the frame helps to keep the pile in motion after it is started by the blow. Generally, the blows follow each other so rapidly that the pile is in continuous motion. The limited vibration thus developed in the pile is also an aid in securing its penetration, particularly in ground containing a large percentage of sand which otherwise offers considerable resistance. The vibration is limited by the weight which constantly rests upon the pile.

3-9. Advantages of Steam-hammers. The following advantages are claimed for the use of the steam-hammer by those who have also had experience with the drop-hammer: (*a*) The pile is held in position and guided more firmly while driving, thus keeping the pile from dodging or getting out of line and avoiding the labor of toggling. (*b*) Serious damage to the pile, such as brooming, splitting, etc., is avoided. Hence piles of softer wood may be employed. (*c*) Extra time and cost for the use of a ring on the pile head are saved. (*d*) The driving is equally effective for any position of the pile head in the leads. (*e*) A pile may be driven several feet (7 or 8 ft. with some hammers) below the bottom of the fixed leads without the use of extension leads. A few feet may often be saved in cutoff by thus driving below the elevation of rail. (*f*) When driving into soft material or into sand, the rapidity of action keeps the pile in motion and prevents the earth from recompacting around the pile until the driving ceases, thus reducing the frictional resistance. (*g*) More piles can be driven in a given time and often with a smaller crew. (*h*) The steam-hammer has been used effectively in places and under conditions where it was found to be impossible to use a drop-hammer successfully. This relates to cases of limited head room as well as to difficult subsurface conditions. (*i*) Less injury is caused to adjacent foundations and less breaking of glass and plastering in adjoining buildings. (*j*) The leads last about three or four times as long as when a drop-hammer is used. (*k*) On track pile drivers less injurious strains are caused in the car and machinery, thus reducing the cost of maintenance. (*l*) Although the first cost of the steam-hammer is much greater, the total cost of driving is reduced.

Exceptional cases have been reported in which a steam-hammer has been unable to force a pile through a hard crust. A drop-

hammer may succeed in such a case because of its heavier blow, but it is more likely to break the pile.

3-10. Rings. It is important to cut off square the butt of a pile, so that the impact of the hammer may be distributed uniformly over the surface. Since the butt tends to change its position slightly in the leads during driving, it has been found advantageous by experience to make the lower surface of the drop-hammer slightly concave. This provision counteracts the tendency toward lateral movement of the pile to some degree. When the pressure on any fibers exceeds their ultimate resistance in compression, they will yield by bending, buckling, or crushing, after their adhesion to adjacent fibers is destroyed. When the fibers are once broken down, every blow of the hammer tends to injure the fibers farther down. As wooden fibers are far more compressible when a force is applied on their sides instead of their ends, the bruised head of the pile thus becomes more elastic and acts somewhat like a spring or cushion. When the height of fall for the drop-hammer exceeds a certain value, a part of its energy is expended in destructive work like that just indicated, leaving less for useful work, reducing its efficiency in forcing the pile to penetrate the ground. This breaking down of the fibers is called "brooming." The fall of the hammer may be so great that nearly all the energy is used up in brooming the pile.

It is often found that no increase in penetration is secured by increasing the fall or drop above 10 to 15 ft. It is possible to estimate approximately the loss of energy due to brooming by comparing the number of blows required per foot of penetration before and after cutting off the broomed top. From the record of a pile driven by a steam-hammer, under the direction of D. J. Whittemore, it was observed that in driving the pile from the twelfth to the twenty-second foot of penetration, 4,682 blows were struck, or an average of 468 blows per foot. Immediately after cutting off the broomed top at two different times, only 275 and 213 blows, respectively, were required to drive the pile the next foot. Their average of 244 blows indicates the number required under the condition of a sound head, and accordingly it appears that on the average only about 52 per cent of the available energy was consumed in securing the penetration of the pile. The loss in this case is considered excessive. The progressive effect of brooming is shown in the following number of blows required for the tenth to the fourteenth foot of penetration, respectively: 73, 109, 153, 259, 684.

The crushing of the fibers is frequently followed by the splitting of the pile head. This tendency is promoted by failing to cut off

enough of the butt as it comes from the forest to cover the entire section area of the pile, for if the hammer hits only one-half of the area it will force that part down into the head and split it.

To prevent splitting and to reduce brooming, the head may be hooped by a pile ring. The sizes range from 2 by $\frac{3}{8}$ to 4 by 1 in. The diameters vary to suit different sizes of pile. They are made of the best quality of wrought iron that can be obtained. Rings of the best bar iron usually last to drive 50 oak piles or 200 cedar piles, those of the best hammered iron for 75 oak piles or 300 cedar piles. Rings made out of old car axles have been used for 250 oak or 6,000 cedar piles.

In fitting the ring the pile is neatly chamfered down at least 5 in. from the end, so that the ring will just catch on; a blow of the hammer puts it into place. To remove the ring, a cant hook or

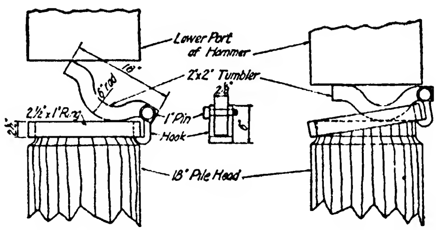


FIG. 3-10*a*.—Mechanical Device for Removing Rings.

peavey is used, the pile line being fastened to its end to apply steam power. If the pile brooms too much in spite of the ring, the recognized remedy is to saw off the broomed part, so as to present a solid surface to the hammer and put the ring on again.

In Fig. 3-10*a* is illustrated an ingenious mechanical device for removing rings. When the pile has been driven, the tumbler or lever is placed on the top of the pile and the hook is caught under the ring. The hammer of the pile driver is then allowed to drop a few inches to loosen the ring.

3-11. Caps. A more effective and less expensive method of protecting the head of a timber pile from brooming and splitting is the use of a pile cap as shown in Fig. 3-11*a*. It consists of a casting with a tapered recess above and below. The chamfered head of the pile fits into the lower recess, and a short cushion block of hard, tough wood is fitted into the upper one. The block is frequently provided with an iron hoop or ring around its top. The cap has jaws on the sides like the hammer which engage the leads; hence the head of the pile is held in position and guided while driving. After the pile is driven, the cap is hooked to the hammer by ropes

and pins and raised with it. While the cap protects the pile head, the short cushion block requires frequent renewal, since it gets the direct impact of the hammer. Sometimes a rope mat is placed on top to protect it. White, live, or swamp oak, rock maple, and blue gum have given good service for cushion blocks.

When both drop- and steam-hammers are used on the same work, it is often found that the drop-hammer causes brooming when the steam-hammer gives no indication of it. In hard driving, however, it becomes important to protect the pile head under steam-hammer action. Sometimes this is done by spiking a flat steel plate on the pile to receive the blow, or a dished or cupped striking plate may be substituted for the flat plate. A better arrangement is adopted for some makes of steam-hammers. The Warrington hammer substitutes for its ordinary base what is known as the McDermid patent base (Fig. 3-11*b*), in which a recess is provided for a thick steel plate inserted through a slot in the side, covered by a door. The plate is held in place by the base and thus avoids the danger to the crew which occurs with the separate flat or dished plate.

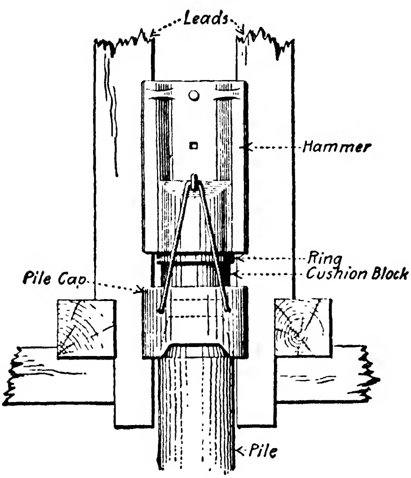


FIG. 3-11*a*.—Casgrain's Pile Cap.

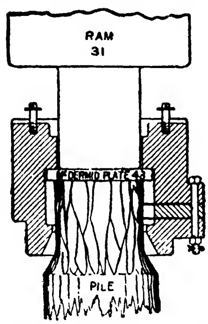


FIG. 3-11*b*.—McDermid Base.

In the Industrial Brownhoist hammer a loose pile-cap casting in the shape of a hollow frustum of a cone enlarged downward and open at the bottom is self-contained in the hammer base. This cap receives the chamfered head of the pile and projects below the hammer base casting until pushed up flush when the weight of the hammer rests on the pile. The McKiernan-Terry and the Union steam-hammers are provided with an anvil block, in the base, which rests on the pile (see Figs. 3-8*c* and 3-8*d*).

3-12. Followers. When a pile has to be driven below the leads, or below the ground or water surface, a follower is generally

employed. A follower is a member interposed between the hammer and a pile to transmit blows to the latter when below the foot of the leads. In its simplest form a follower may consist of a short pile or stick of white oak of the requisite length and diameter. To keep its lower end in position on the pile, a follower band may be used which is flared both upward and downward. However, it is better to use a follower cap. This is a cylindrical casting with a horizontal diaphragm at the middle, which is bolted to the lower end of the timber follower and fits over the head of the pile. The upper end of the follower is held in position by the recessed base of the steam-hammer or by a pile cap if a drop-hammer is in use.

A better kind of follower consists of an extra strong pipe cast into the follower base, so as to avoid the objections to the use of bolts. A stick of turned hardwood is driven into the pipe. An iron band is shrunk on the pipe so as to project beyond the top into which is fitted a hooped oak driving block that may be replaced when worn out. Patented followers are also used. To these followers pipes are attached by which steam or air may be introduced on top of the pile to release the follower when such aid is needed in certain soils. When followers are used to drive piles through a considerable depth of water, the base of the follower should engage extension leads so as to hold and guide the head of the pile properly. In deep water with a swift current it may not be possible to handle the follower effectively. In such cases long piles are driven while their heads remain above the surface; afterward they are cut off at the proper elevation.

In placing piles for a bridge over the Maumee River in Toledo, the piles, 19 to 25 ft. in length, were placed in 35 ft. of water by first lowering a 19-in. steel tube 45 ft. long to the blue clay, through which piles were driven by means of a follower. The bottom of the tube extended a few inches into the clay, while the top was held in the leads. The 46-ft. timber follower was just large enough to fit neatly inside the tube, and it had a steel sleeve at the bottom which encased 18 in. of the top of the pile and held it true.

A follower generally absorbs a considerable percentage of the energy of the hammer, frequently amounting to 50 per cent. The loss is greater when the lower end of the follower is not guided by the leads and the pile is set into unusual vibration. The following record by J. E. Crawford shows, however, that under proper conditions there may be no appreciable loss in the effect of the blow. The pile sank of its own weight 6 ft., then the hammer with its housing weighing 6,000 lb. was put on it, and it sank 5 ft. farther. The

number of blows for each succeeding foot of penetration were 9, 5, 13, 20, 14, 16, 17, 15, 30, 40, 47, 65, 45, 26, 22, 33, 60, 55, and 55. The follower was put on and the number of blows required per foot were 55, 75, 56, 60, 73, 90, 113, 115, and 102 blows for the last 7 in., giving the pile a penetration of 39 ft. 7 in.

The type of hammer shown in Fig. 3-8*c* drives piles below the water surface without the aid of a follower. To keep water out of the lower cylinder, compressed air is supplied through an air hose connected to the manhole cover. The exhaust from the hammer is carried to the surface through a hose. The hammer is held in place by extension leads. Piles have been driven with a hammer submergence of 70 ft. or more.

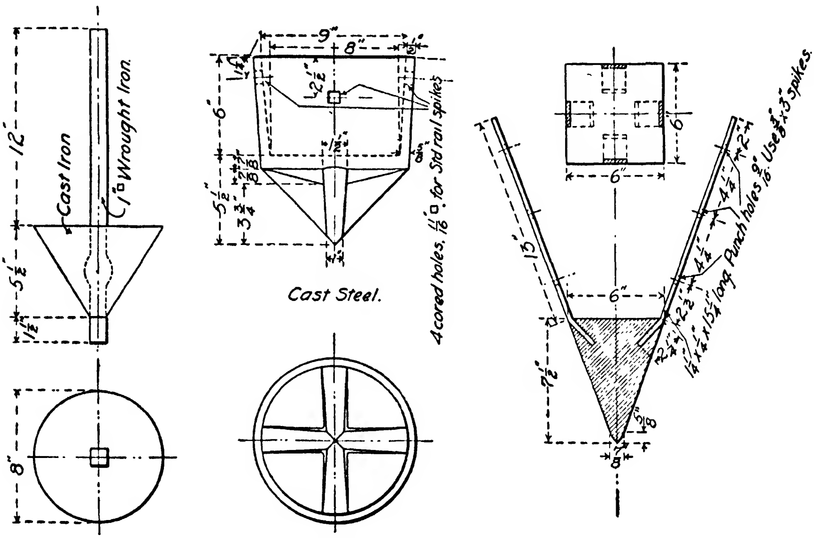
3-13. Points and Shoes. The foot of a timber pile should always be cut off perpendicular to its axis, since this facilitates driving it true to line or position. In soft and silty ground, or where the driving is easy, it is not necessary to sharpen or point the pile. If a pile penetrates soft material and rests upon a hard stratum, thus acting as a column, the unpointed foot has the additional advantage of providing a larger bearing area. The blunt end on striking a root or any small obstruction will generally break the obstruction without deflecting the pile.

In driving a pile with a blunt end, a cone of compressed earth forms under it and acts in most respects as if the pile were pointed. It is frequently claimed that even in driving through hard material a blunt pile will keep more nearly to the required position than if it is pointed. This implies that the cone of earth is more likely to have the form of a fairly good cone or pyramid than the wooden point made by sharpening the pile. Such a contention can hardly be maintained if the pointing is properly done. When coarse gravel or boulders are encountered which destroy the cone of compact earth, crush the fibers of the timber, and wedge them apart, it is desirable to reduce the area of the foot by pointing. In general, when the ground is at least moderately compressible and the driving is not hard, the foot of the pile may be left unpointed.

When the driving is hard for most of the penetration, as in stiff clay or in material that is but slightly compressible and hence must be displaced, it is advisable to point the pile, so that it may separate the material at the foot like a wedge. In pointing a pile it is preferably sharpened to the form of a truncated pyramid, the end being from 4 to 6 in. square. If the end is too small the fibers lack the necessary strength to resist brooming. The length of the point may be from one and a half to two times the diameter of the foot.

Another advantage of pointing is to increase the rate of penetration, or to reduce the energy required. In compact material the bearing power of a pile is practically the same with or without the point. Experience has also shown that piles with pointed ends may be successfully driven through old timber cribwork while attempting to drive them with blunt ends resulted in broomed tips and split and broomed heads.

Sometimes the timber point is replaced or protected by a metal shoe. Figure 3-13*a* shows an undesirable form which tends to split the pile when the side of the shoe strikes an obstruction. Figures

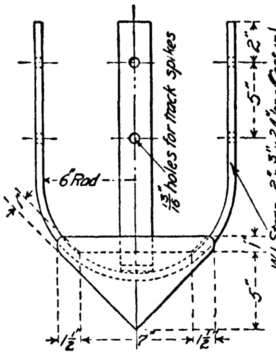


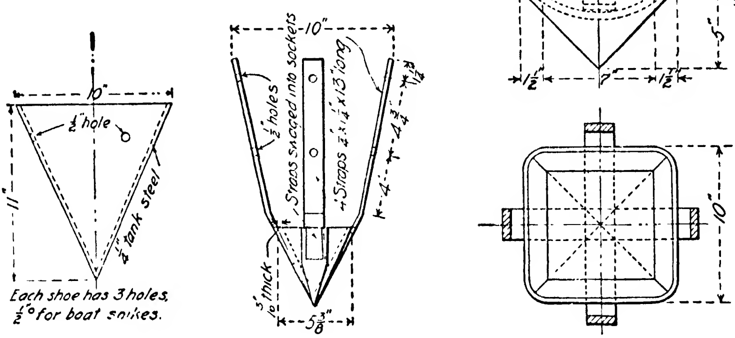
FIGS. 3-13*a*, *b*, and *c*.—Shoes for Timber Piles.

3-13*b*, *c*, and *f* illustrate the best forms, since the timber has a square bearing on the upper flat surface of the shoe and the sides of the socket or the straps permit such a firm fastening as to make the shoe act like an integral part of the pile. Those in Figs. 3-13*d* and *e* are not quite so effective unless a close fit is secured in the socket at an increased labor cost. Shoes are used by some engineers when piles are driven into material containing boulders, riprap, coarse gravel, shale, slate, hardpan, buried timber, very hard clay, and coral rock. Another use is to penetrate a thin, hard stratum (2 ft. or less) which overlies a softer one. They are also attached to piles for bridge falsework in order to gain a foothold on rock bottom. In one case it was thus possible to secure sufficient penetration to hold the piles against a 20-ft. rise in the river and a swift current.

On the Key West Extension of the Florida East Coast Railway, where numerous pile foundations are built on coral rock containing pockets of different sizes, a hole was made by driving a steel punch with the pile-hammer and then driving in the timber pile with a few light blows. In order to permit the punch to be withdrawn readily, it was provided with a foot slightly larger in diameter than its body.

Some engineers and contractors condemn the use of shoes unqualifiedly because of personal unsatisfactory experience. But in many cases such experience is probably due to employing shoes which were improperly designed or constructed,





FIGS. 3-13*d*, *e*, and *f*.—Shoes for Timber Piles.

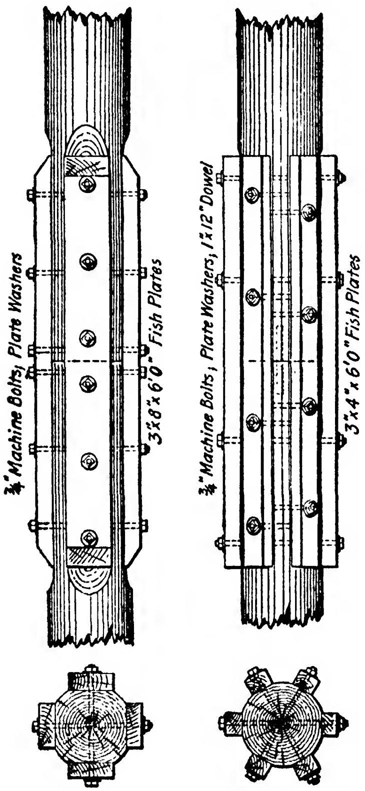
whereas in others the piles should have been omitted, since the ground was hard enough to support the substructure directly.

3-14. Splices. It is occasionally necessary to use longer piles than can be obtained in single sticks. It becomes necessary, therefore, to splice two piles together end to end. For this purpose a fishplate joint is usually the best, since it provides lateral resistance. Either four or six timber fishplates are usually used, as illustrated in Figs. 3-14*a* and *b*. Wrought-iron or steel fishplates may be employed instead of wooden ones, thus reducing the sectional area at the joint. Another method is to use a metal sleeve consisting of a piece of pipe, as indicated in Fig. 3-14*c*. Half-lapped joints fastened with either bolts, bands, or wire wrapping are sometimes used, but they are deficient in lateral strength and stiffness.

For the falsework to erect the Poughkeepsie bridge, where the water was 55 ft. deep, 55-ft. piles were spliced to 75 ft. piles by means of fishplates 20 ft. long, eight fishplates 4 by 5 in. in section being

fastened to the piles with ½-in. wrought-iron spikes 8 in. long. On a job in New Orleans where 40-ft. creosoted piling was spliced on top of 60-ft. untreated piling, the spliced ends of the piles were shaped to a hexagon and six 4- by 6-in. fishplates 20 ft. long bolted and spiked to the piles. These bolts were ¾ in. in diameter and spaced 2½ ft. centers, ⅝- by 10-in. spikes being driven midway between the bolts. In addition, four heavy steel bands in halves were clamped on to further hold the fishplates in place. Tests indicated that the spliced section was stronger in cross-bending than sections outside of the splice.

In splicing piles for the foundation of a Post Office Department building in San Francisco, where the length ranged up to 150 ft., splices were made by using steel pipe 10 in. in diameter, ¼ in. thick, and 18 in. long. A 1- by 18-in. dowel and a 4-in. square connector grid were also used.



Figs. 3-14*a* and *b*.—Fishplate Splices for Timber Piles.

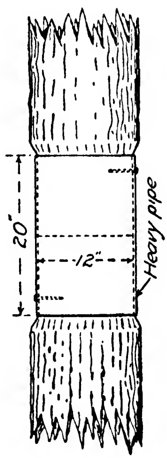


Fig. 3-14*c*.—Tubular Splice for Timber Pile.

Pile splices may also be required where piles have to be driven in sections on account of limited clearance, as under a bridge. In pile trestles where the upper portions of long piles are decayed, repairs may be made by cutting out the decayed section and inserting new timbers. In one case four steel angles were used as fishplates for each pile. They were well fastened with spikes, and each

end of the joint was wrapped with a band of heavy wire, spiral wrapping extending between them. A shell of concrete was then cast around the joint to protect the metal.

3-15. Lagged Piles. A lagged pile has pieces of timber bolted around the sides of the pile in order to increase its bearing power. It increases the area of cross section and also the surface of the sides of the pile, which is of more importance since such piles are used only in soft material. The New York City Docks Department made a test in 1902 of the relative bearing capacity of lagged and unlagged piles driven in North River mud, the results of which are recorded in the *Transactions of the American Society of Civil Engineers*.[1] The discussion by the author of the paper implies that the ultimate bearing capacity was increased about 50 per cent. The total penetration of the piles ranged from 47.1 to 52.6 ft., while the lagging was only 30 ft. in length. Although it is not stated what position vertically the lagging occupied, it appears that the surface in contact with the mud was increased about 70 per cent.

In some tests made in 1915 with a cluster of piles spaced $2\frac{1}{2}$ ft. centers and with alternate piles lagged, the settlement under a total load of 230 tons was $11\frac{1}{4}$ in. in 280 days, whereas, in a cluster in which all the piles were lagged and spaced $3\frac{1}{4}$ ft. in one direction and $3\frac{1}{2}$ ft. in the other, the settlement was $8\frac{1}{4}$ in. For the first 60 days the settlement was about the same for both clusters. The lagging consisted of four 5- by 6-in. sticks 30 ft. long, spaced around the four quarters of the circumference and at the lower end of the pile. The total embedment of the piles below ground was 65 ft.

[1] Vol. 54*F*, pp. 8, 27, 1905.

CHAPTER IV

DRIVING AND PROTECTING TIMBER PILES

4-1. Theoretical Considerations. A brief description is given in Art. 3-5 of the energy transformations that occur when the hammer strikes the pile. Of the total kinetic energy possessed by the drop-hammer (or ram of the steam-hammer) at the instant of impact, a certain proportion is lost in heating and brooming the head of the pile (or the cap), a certain proportion remains in the hammer in the form of kinetic energy, and the remainder goes into the pile.

Let W_h = weight of drop-hammer or ram of steam-hammer
W_p = weight of pile and pile cap
v_h = velocity of hammer prior to impact
v_p = velocity of pile (= 0) prior to impact
v'_h = velocity of hammer following impact
v'_p = velocity of pile following impact
H = fall of hammer
e = efficiency of hammer
r = W_h/W_p

The energy of the hammer at the instant of impact is

$$W_h \frac{v_h^2}{2g} = eW_hH.$$

By the principle of conservation of momentum (see any standard text on mechanics) we have

$$W_hv_h + W_pv_p = W_hv'_h + W_pv'_p. \tag{4-1a}$$

The elastic properties of bodies are indicated by the *coefficient of restitution*, *n*. For fully elastic bodies $n = 1$, and for completely inelastic bodies $n = 0$. This coefficient is defined as the ratio of the velocity of one body with respect to another body after impact to the relative velocity before impact; or, in our case,

$$n = \frac{v'_p - v'_h}{v_h - v_p} \tag{4-1b}$$

From these two equations we get, noting that $v_p = 0$,

$$v'_p = \frac{W_h v_h(1 + n)}{W_h + W_p} \qquad \text{and} \qquad v'_h = \frac{v_h(W_h - W_p n)}{W_h + W_p}.$$

The kinetic energy of the pile after impact is

$$\text{K.E.}_p = \frac{W_p v'^2_p}{2g} = \frac{W_h v_h^2}{2g} K_1 = eW_h H K_1, \tag{4-1c}$$

where $K_1 = \dfrac{r(1 + n)^2}{(r + 1)^2}$.

The kinetic energy left in the hammer after impact is

$$\text{K.E.}_h = \frac{W_h v'^2_h}{2g} = \frac{W_h v_h^2}{2g} K_2 = eW_h H K_2, \tag{4-1d}$$

where $K_2 = \dfrac{(r - n)^2}{(r + 1)^2}$.

The kinetic energy lost in heating and brooming the top of the pile (or cap) is

$$\text{K.E.}_l = eW_h H - eW_h H K_1 - eW_h H K_2 = eW_h H K_3 \tag{4-1e}$$

where $K_3 = \dfrac{(1 - n^2)}{r + 1}$

It will be observed that K_1, K_2, and K_3 are ratios. For example, K_1 is the ratio of the energy immediately transmitted to the pile to the original energy of the hammer. The energy distribution for several ratios of hammer weight to pile weight and for three values of n are given on Table 4-1*a*.

From this table it appears that the maximum percentage of available energy is immediately transferred to the pile when $W_h = W_p$, which fact may also be demonstrated by differentiating K_1 with respect to r and equating to zero. It will be seen that, the heavier the hammer with respect to the weight of the pile plus cap, the less the energy lost in heat. On the other hand with the heavier hammers the energy remaining in the hammer increases, and the over-all efficiency depends on what proportion of this residual hammer energy is later transferred to the pile. The residual velocity of the hammer, v'_h, becomes zero when $W_h = W_p n$, as will be noted from the equation for v'_h. When W_h is less than $W_p n$, the velocity of the hammer after impact will be negative, or upward, in which case it is evident that its energy is not available to the pile.

When W_h is greater than $W_p n$, the velocity of the hammer will be downward but less than that of the pile. However, the elastic

Table 4-1*a*

$r = \frac{W_h}{W_p}$	n	Proportion of total energy: Transferred to pile $K_1 = \frac{r(1+n)^2}{(r+1)^2}$	Remaining in hammer $K_2 = \frac{(r-n)^2}{(r+1)^2}$	Lost in heat $K_3 = \frac{(1-n^2)}{r+1}$
0.25	0.2	0 23	0.0	0.77
0.50	0.2	0 32	0.04	0.64
0.75	0.2	0.35	0.10	0.55
1.00	0.2	0.36	0.16	0.48
1.50	0.2	0.35	0.27	0.38
2.00	0.2	0 32	0.36	0.32
0.25	0.4	0.31	0 01	0.68
0.50	0 4	0 44	0 0	0.56
0.75	0 4	0.48	0 04	0.48
1.00	0.4	0.49	0 09	0 42
1.50	0.4	0 47	0 19	0 34
2.00	0.4	0.44	0 28	0.28
0.25	0 6	0 41	0.08	0 51
0.50	0 6	0 57	0.0	0.43
0.75	0 6	0 63	0 01	0.36
1 00	0.6	0.64	0 04	0 32
1.50	0.6	0 61	0.13	0.26
2.00	0.6	0 57	0.22	0.21

recovery of the pile keeps the hammer in contact with the top of the pile, and so most of the residual hammer energy is probably transmitted to the pile. But it is doubtful if any useful purpose is served by having the weight of the hammer materially greater than that of the pile plus cap.

The energy transmitted to the pile is used in part in compressing elastically the pile, the earth around the pile, and perhaps the pile cap; and in part it is used in forcing a penetration of the pile against the resistance of the earth, the last being the object of the pile driving operations. For a further discussion of this subject, as well as for values of *e* and *n*, see Art. 5-3.

4-2. Observations in Practice. In driving piles a wide variety of phenomena may be observed depending on the type of soil and its density. When piles are driven into some sands, such as fine-grained quicksands, the surface of the ground may subside as much as a foot. This indicates that the reduction of voids resulting from

the vibrating action of the pile driver must have been large since the total reduction in volume includes the volume of the piles plus the volume of subsidence. Such an action involves a considerable increase in the bearing power of the sand.

On the other hand, when piles are driven into soft clay, the surface of the ground between the piles may rise a number of inches, this indicating that there has been little or no compaction of the soil. In fact the soil may become softer and possess less bearing power since the compressive strength of an undisturbed sample of clay is always greater than that of a remolded sample having the same water content (Art. 2-8).

In normal soils the resistance to driving increases with the depth, due primarily to the increase in the circumferential area subject to frictional resistance, and secondarily to the increase in soil density.

The great variety of experiences which may occur is illustrated by those encountered in driving piles for the Ogden-Lucien Cut-off of the Central Pacific Railway. The nature of the bottom of Great Salt Lake was found to be so variable that at times a blow of the hammer drove a pile only 1 or 2 in., and at other times one or two feet. Again a pile seemed to strike a hard stratum and refused to sink farther under many blows, but after being forced through, the pile sank as much as 2 or 3 ft. per blow. Frequently a pile with a penetration of 30 to 50 ft. would suddenly rise 2 or 3 ft. during a short delay of the hammer. At the end of the temporary trestle, to be later replaced by a rock fill, a new difficulty was encountered. The first pile, 26 ft. long, was driven out of sight by a single blow and when another pile, 28 ft. long, was placed on top of it, the next blow of the hammer sent both out of sight. The formation was found to be a deep mud deposit due to the Bear River. As the mud was 50 ft. deep, two 40-ft. piles were driven on top of each other. The trestle supported by these spliced piles carried the trains until the rock fill was completed and settled.

In certain kinds of clay the lateral spring of a pile under the hammer blows makes a hole slightly larger than the diameter of the pile, allowing surface water to find its way to the foot of the pile, thus reducing both the skin friction and the bearing capacity of the clay under the foot of the pile. This action explains why cases have been observed where piles settled under moving trains after a rain, although the resistance of the pile when driven was considered satisfactory.

In some soils a pile may sink some distance and then refuse to go farther, but it will resume penetration after an interval of rest

(see Art. 5-6). The driving of one pile may cause adjacent piles to rise and in soft ground or mud often causes an adjacent pile previously driven to move away slightly.

If it is desired to compact the ground in a given area uniformly, it is best to begin driving piles at the center and work outward to the perimeter. If the order of procedure is reversed, the driving becomes more and more difficult, and to secure uniform penetration the outer piles will be forced to rise more or less.

4-3. Weight and Fall of Hammers. A committee of the American Railway Engineering Association[1] recommended 3,000 lb. as the minimum weight of a gravity-hammer for general railroad service. In hard driving, experience has frequently proved that piles can be successfully driven with a 4,000-lb. hammer when a 2,000-lb. hammer fails to do so. In one case where the weight of the hammer was only 56 per cent of the weight of the pile, the piles failed to sustain without settlement one-half the load they were designed to carry, showing that the inertia of the pile absorbed most of the energy.

In the best practice the height of fall for a drop-hammer is limited to about 20 ft. Although a fall of 5 ft. or less may be used in soft ground, its value will most generally range from 10 to 15 ft. If occasionally a fall exceeding 20 ft. is used to penetrate a hard stratum, it should not be continued long on the same pile for fear of damaging the pile. A low fall, or short drop, has the additional advantage of securing a more rapid succession of blows, which in most kinds of earth is advantageous in securing penetration and thus economizing time.

If W_h denotes the weight of the hammer in pounds, and H the height of its fall in feet, W_hH will represent closely the work done by the hammer in a single blow for free fall. It is considered that 30,000 ft.-lb. is about as small a value for W_hH as it is economical to use on work of considerable magnitude and that 50,000 ft.-lb. should rarely be exceeded on account of the limited strength of timber.

The only drawback that may be alleged against a heavy hammer is the increased capacity required for the hoisting engine and equipment, but in work of any magnitude it is economical to provide the equipment required to do the work expeditiously and well. If the fall is too low, then the rate of penetration of the pile is unduly reduced; and on the other hand, if the fall is too high, the effect is analogous to that of a bullet, tending to unduly damage the pile. The permissible height of fall should be adjusted to the resilience of

[1] *Proc. Am. Railway Eng. Assoc.*, vol. 41, p. 330, 1940.

the timber composing the pile, as well as to some of its other qualities, including the strength in tension across the fiber, which measures resistance to splitting.

When the rope is fastened to a drop-hammer and the falling hammer must pull the rope with it and thereby also revolve the drum of the hoisting engine, the effective fall is very materially decreased. The following table gives the penetrations of three piles, driven by different pile drivers under conditions of both free and restrained fall:

Pile number	Weight of hammer, pounds	Height of fall, feet	Penetration, feet	
			Restrained fall	Free fall
1	2,470	40	0.5	0.7
2	2,750	45	0.7	0.9
3	2,500	46	0.32	0.4

The above noted committee also recommends that steam-hammers used for driving timber piles should have a weight of the striking part not less than the weight of the pile. Where hard driving is encountered, it may be necessary to reduce the length of the stroke to avoid injuring the pile.

4-4. Driving Piles Butt Down. It is the general practice to drive piles with the tip downward. Occasionally, however, special conditions make it advisable to drive them with the butt downward. It has been found difficult at times to keep a pile down after being struck by the hammer, the pile beginning at once to rise, lifting the hammer with it. Upon raising the hammer, the pile may shoot upward 5 ft. or more, or the pile may exhibit this tendency but slightly when driven, but the following morning will stand with its head a number of feet higher than before. This behavior is ascribed to a substratum of quicksand, and the difficulty is usually overcome by driving the pile "butt down."

Another condition occurs when piles are driven through very soft ground and the load has nearly all to be borne by the foot. The substratum may require the larger bearing area afforded by the butt of the pile to carry the load. Some engineers recommend that tall pile trestles which are to be filled should have the piles driven butt down, thus leaving no hollows to cause trouble as the embankment settles. The bracing can be removed as the filling rises. In hard material the butt may have to be pointed to a smaller diameter to facilitate penetration. Great care must be

exercised in driving on account of the smaller area of the tip, which receives the blow; the smaller percentage of heartwood in that area; and the weaker fibers of the wood which grows in the upper part of a tree trunk.

In some cofferdam construction on the Ohio River where it was necessary to drive about 600 oak guide piles into hard gravel, it was found that the best way to secure adequate penetration was to drive them with the butts down. In this manner the resistance encountered due to the wedge action of piles as usually driven was avoided, and the useful effect of the blow was all transmitted to the foot of the pile.

An interesting example relates to piles driven 4 to 6 ft. apart both ways in the embankment of the Yazoo Canal near Vicksburg, Miss., to stop the bank from sliding on the adjacent railroad track during the low-water stage of the river. By driving the piles butt down, advantage was taken of the larger cross section at the lower elevation where the bending moment was a maximum. Pile trestles have resisted the pressure of ice going out by having extra flexural strength due to the piles being driven butt down. When piles have to be driven into sand with their butts down, the water-jet should be employed (Art. 4-6).

4-5. Driving Batter Piles. A batter pile is a pile driven at an inclination to resist forces which are not vertical. It is sometimes called a "spur pile." When a pile driver is designed to drive batter piles as well as the ordinary vertical piles, its leads are suspended from a horizontal pin to permit them to be swung laterally like a pendulum. Hence they are known as swinging leads and sometimes as pendulum leads. The pivot is attached to the top of a tower, the front timbers of which are inclined laterally to provide the requisite transverse bracing. Figure 4-5*a* shows batter piles being driven in the trestle approach of the Dumbarton bridge across San Francisco Bay. Although this illustration is that of a track driver, the arrangement of swinging leads shown is the same as for ordinary land drivers or for floating pile drivers.

Occasionally timber drivers are arranged to drive batter piles by having a removable section at the bottom of the backstays, so that the tower revolves backward about hinges located near the foot of the leads. Another scheme consists in taking a separate set of leads and temporarily bracing them to the tower of a pile driver. In this manner, batter piles 60 to 70 ft. long were driven for car-dump foundations at the Erie Railroad dock at Cleveland, the piles sloping downward toward the river.

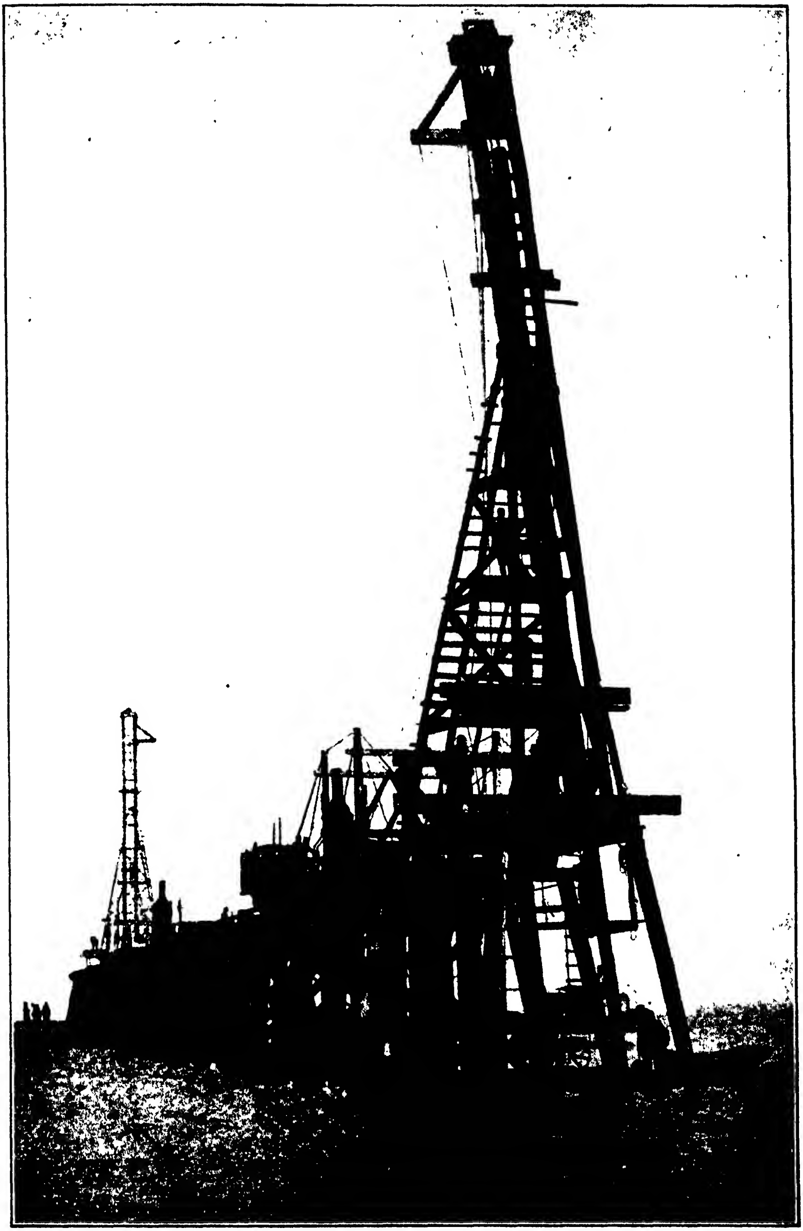


FIG. 4-5*a*.—Driving Batter Piles on the Trestle Approach to the Central California Railway Bridge over San Francisco Bay at Dumbarton Point, August 14, 1907.

Steel pile drivers are almost always built to drive batter piles as well as vertical ones (see Figs. 3-6*c*, 3-6*d*, 6-7*c*, and 7-4*a*). Most of them can drive piles with a wide range of batters. The one illustrated in Fig. 7-4*a*, probably the heaviest driver ever built, can drive on a maximum batter of 1 in 4.

4-6. Use of the Water-jet. A method of placing a pile in position, which differs radically from that of driving it with a hammer, consists in displacing the material by means of one or more jets which discharge water under pressure at or near the foot of the pile. As the water comes up around the pile carrying with it some of the material, it also diminishes the frictional resistance of the pile. In some kinds of earth the hammer is merely placed on top of the pile to increase the pressure by its weight, while in other cases the hammer is operated with a restricted fall to secure a greater rate of penetration.

In soft ground, one jet may answer the purpose, but in most cases two jets, used on opposite sides of the pile, give better results. As the pile tends to move toward the side where the jet is operated, the use of two jets usually enables a pile to be placed more accurately in position. Sometimes a third jet is employed, discharging at a higher elevation than the others, if difficulty is experienced in keeping the ground from packing against the sides of the pile.

In order to secure better bearing at the foot, it is customary to shut off the water just before the pile reaches its intended total penetration and to complete the sinking by use of the hammer. This procedure presses the pile firmly into the softened earth and tends to avoid any arching action of the earth that might prevent the material from filling every cavity when it settles into place.

The water-jet may be used advantageously in any material that will settle around the pile after the flow of water ceases. The best results are obtained in pure ocean or river sand. In this material the simplest form of jet may be used, only a moderate pressure is required, a single jet will generally answer, the time of sinking is very short, and the sand packs quickly after the water is shut off, while no blows of the hammer are needed except for the purpose stated in the preceding paragraph. It is fortunate that this is the case, for pure sand offers very high resistance to a pile when driven with the hammer alone, especially with a drop-hammer. Even in quicksand, this is frequently found to be true. With the jet a pile may be sunk in sand without danger of injury, whereas it is difficult to avoid injuring piles when driven into sand without the aid of the

jet. Where jets are not used more time and a larger expenditure of energy are required.

Piles have been driven with the aid of the water-jet process in mixtures of sand and silt or gravel, if the latter is not too coarse; in loam, clay, marl and even "gumbo" in pockets, although a special nozzle is required for the material named last. Some of the most experienced engineers in the use of the jet have driven piles with its aid in "sand and clay and the hardest kind of bottom," and in "almost any material except hardpan and rock." In hard ground, the jet process may be used advantageously in case sufficient volume and pressure of water be provided. In clay it may be economical to bore several holes in the earth before driving the pile, thus securing the accurate location of the pile and its lubrication while being driven. Where the material is of such a porous character that the water from the jets may be dissipated and fail to come up in the immediate vicinity of the pile, the utility of the jet process is uncertain except for a part of the penetration. In mixtures with gravel or coarse material the water will often wash out the sand and finer material leaving the stones in the hole to interfere with the penetration of the pile. This action may often be remedied, however, by increasing the volume and pressure of the water.

For driving in sand, the jet should be hung on a rope passing over a pulley in the driver so that it may be kept moving up and down with its point near the point of the pile. If this is not done, the pipe is likely to "freeze" fast and cannot be moved. After the pile reaches a depth of 10 or 15 ft., the water will sometimes fail to come up around it, breaking out on the surface at a considerable distance, perhaps around a pile driven previously. When this occurs, it indicates that the jet has not been kept moving sufficiently, or an auxiliary jet discharging at some intermediate depth may be needed. In any case the jet should be withdrawn at once and immediately put down again, thus usually reestablishing the flow of water along the pile. Where piles are sunk 20 ft. or more into sand, it is advisable to have two jets. One is to be kept moving with its nozzle slightly ahead of the pile, while the other is slowly raised and lowered between the foot of the pile and the surface to maintain the flow along the pile. On the other hand, when the material is soft and readily compressible as in silt, or in fine sand mixed with silt or a small percentage of clay, it may not be economical to use a jet, since the pile may be driven quickly without risk of injury by means of a steam-hammer.

In general, the water-jet should not be attached to the pile, but handled separately. The nozzle is usually extended a small distance, not exceeding a foot, below the foot of the pile. Sometimes, however, it is necessary to move it up and down to reduce the frictional resistance on the pipe, or to change its position if a boulder is encountered, so as to excavate an opening into which the boulder may be pushed by the pile. If this is not sufficient to displace an obstruction, the pile may be raised a little and dropped with the hammer resting upon it. It is not desirable to bend the pipe, as is sometimes done just above the nozzle. For depths not exceeding 15 to 20 ft. and when the ground does not consist of layers differing materially in character, the average rate of penetration is often found to be remarkably uniform, independently of the depth.

In Florida, palmetto piles are sometimes used, since they are comparatively free from the ravages of the teredo. This wood has a hard shell and a soft interior and cannot stand heavy blows with a hammer. Such piles may be easily sunk into hard sand by a water-jet, the hammer resting on top and occasionally tapping the pile, the fall being only 3 to 6 in. By keeping the pile and jet pipes constantly moving, the sand is kept from closing in on the pile until it occupies its final position. With the aid of the jet, piles may be sunk as readily with the butt down as with the tip down.

In using the water-jet, the quantity of water should be ample. In most cases volume rather than velocity is necessary. The velocity must be sufficient to excavate the sand below the foot of the pile and to make it "live" or "quick," while the volume is large enough to force the water to escape by rising along the sides of the pile to bring the material to the surface, and at the same time to reduce the surface friction, if it does not entirely eliminate it. In beach sand, piles have been jetted down within 18 in. of adjacent piles without disturbing them, showing that in this material the movement of the water is confined to a small radius horizontally. In cities the water-jet cannot be used so freely as elsewhere on account of the danger of settlement to adjacent foundations and injury to the heavy structures supported by them.

The earliest authenticated use of the water-jet in sinking piles appears to have been introduced on the construction of a wharf at Decrow's Point, Matagorda Bay, Texas, in 1852, and to have arisen from a suggestion made by Lieut. George B. McClellan, Corps of Engineers, U. S. A. The water was pumped by an ordinary hand pump through a rubber hose with a gas-pipe nozzle, the nozzle being placed close to the tip of the pile. The historical develop-

ment of the water-jet process is described at length in an article on The Water-Jet as an Aid to Engineering Construction, by L. Y. Schermerhorn, in *Proceedings of the Engineer's Club of Philadelphia*, vol. 17, 1900.

4-7. Equipment for Water-jet Process. The water-jet consists generally of a straight pipe with a nozzle at its end, connected by some length of flexible hose to the discharge pipe from the pump which provides the water under pressure. The suction pipe connects the pump with the source of water supply. The pump is operated by steam, by electricity, or by internal combustion engines. The pipe can be raised and lowered by a line attached to the top leading over a snatch block to be operated by hand power on the ground, or to a spool on the hoisting engine.

The diameter of the jet pipe is either 2 or $2\frac{1}{2}$ in. The discharge pipe of the pump is in most cases 4 in. in diameter; the diameter of the suction pipe is 6 in. To increase the velocity of the water and thus increase its power to loosen the earth, the size of the pipe is drawn down at the end to form a nozzle. The nozzle is usually circular in section, and its diameter varies from $\frac{3}{4}$ to $\frac{1}{2}$ in. In a few cases a rose jet has been employed, the nozzle having one central opening at the end and five openings around the sides with their axes inclined about 45 deg. to that of the axis of the pipe (see Fig. 4-7*a*). Another form of nozzle is made by flattening the end of the pipe until the opening is reduced to $\frac{1}{4}$ in. This nozzle has given better results than a round one, especially in stiff material, it being rotated back and forth about its axis.

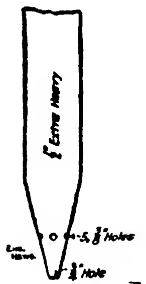


FIG. 4-7*a*. Tip of Water-jet.

The quantity of water to be discharged varies from 50 to 250 gal. per min. It must be sufficient to bring to the surface the material which is next to the pile. The pressure ranges from 65 to 200 lb. per sq. in., although the hose and fittings are sometimes designed to resist a pressure of 250 lb. per sq. in. The higher pressures are required especially when gravel and boulders are encountered. The error most frequently made is to use pumps of insufficient capacity, leading to ineffective work with the jet and loss of time. Unsatisfactory results with the water-jet, due to inadequate and inefficient equipment, explain why the water-jet process has not come into more extensive use in pile-driving practice. In general the plant capacity should be sufficient to deliver at least 100 lb. per sq. in. at two $\frac{3}{4}$-in. nozzles.

A device has been invented by which the jet pipe is handled by means of the water pressure, thus reducing hand labor to a minimum. By turning a valve, the operator who guides the pipe near the pile can control the direction and pressure of the water so as to raise or lower the jet or to hold it in any given position. The jet pipe acts as a plunger inside of a larger pipe which acts as a cylinder, the latter being suspended from a derrick pile driver.

4-8. Preboring Holes for Timber Piles. Experience has shown that, where driving is difficult, it may be more economical to prebore the holes. This was done in placing a pile foundation for a new municipal auditorium at Long Beach, Calif., in 1932, where the holes were drilled by a rotary-drilling equipment similar to that used in oil-well work. The time required for placing the piles was reduced one-half.[1]

The original pile-driving equipment consisted of a 5,000-lb. steam-hammer and jetting equipment. Difficulty was experienced in penetrating a hard clay stratum underlying about 25 ft. of black sand. The contractor developed the novel scheme of equipping the driving rig with a small power-driven rotary table, that gripped a drill stem composed of piping, to the lower end of which was attached a 12-in. fishtail bit. By a rotating motion the bit cut through and pulverized the soil. Water was pumped down through the hollow stem and, as it came out under the bit, washed up the loosened soil along the outside of the stem.

The procedure was to first drill a hole through the hard clay stratum to within a few feet of the required depth, then pull the drill, move the table back from the leads, and drop the pile into the hole before the sand had a chance to refill the hole. The pile was driven for the last 6 or 8 ft.

In applying a test load of 59.5 tons, the design load being 25 tons, settlement amounted to about $\frac{1}{3}$ in., which was completely recovered on removal of the load. This was practically the same settlement that took place on piles that had been driven.

In general, drilling should not be resorted to where driving is not too difficult. The diameter of the drilled hole should not exceed the diameter of the pile tip by more than 1 in. Drilling should be continued only through the hard strata, and the final penetration should be by driving.

4-9. Overdriving Piles. Construction work is full of examples of overdriven piles. Some characteristic types of failure due to this cause are shown in Fig. 4-9*a*, among which are the following:

[1] See *Eng. News-Record*, vol. 109, p. 129, Aug. 4, 1932.

(*a*) brooming of the foot of the pile, as shown in the lower right photograph; (*b*) twisting of the bottom of the pile into a horizontal position as shown in the upper photographs; and (*c*) shear failure as shown in the middle photograph.

A contractor, having noticed the apparently injurious effect on piles due to very heavy blows, ordered experiments to be made to learn just what damage was done and the proper remedy. Spruce and yellow pine piles 40 to 50 ft. long, 12 to 15 in. at the butt, and 6 to 8 in. at the tip were driven into a mixture of clay, sand, gravel, and small cobble stones, which offered a gradually increasing resistance to penetration. A drop-hammer weighing 3,000 lb. was used with the hoisting rope attached. The object was to determine what height of fall could be safely adopted without impairing the integrity of the pile either by brooming its head or its foot, or by breaking it at some intermediate point. The first piles were driven according to the former custom of raising the hammer to the top of the leads or about 25 ft. The fall was gradually diminished as successive piles were driven. Each pile was pulled up by a 100-ton derrick and examined. Nearly every pile driven with a fall exceeding 10 ft. was found to be more or less injured, either by badly brooming at the foot or by breaking at some distance higher. A fall of 10 ft. could be depended upon not to injure the pile.

In 1930 it became necessary to place a new foundation (Art. 17-8) under a 20-year-old 14-story building in San Francisco. The building was supported on 1,100 untreated timber piles. The trouble was caused in part by decay of the pile tops owing to a lowering of the ground-water level and in part by overdriving. The foundation material consisted of about 10 ft. of fine beach sand and 7 ft. of blue mud and shells resting on a hard sandy clay. Investigation made by exposing only the upper 6 ft. of piles supposed to be from 20 to 25 ft. long showed that approximately 25 per cent of the piles were seriously damaged by overdriving. Many of the piles had penetrated only about 6 ft. of the sand, the fibers being badly mashed and sheared. In other cases the lower ends of the piles were found in a horizontal position at the top of the hard stratum.

There is danger from overdriving when the hammer begins to bounce. Overdriving is also indicated by the bending, kicking, or staggering of the pile. When a pile has not penetrated very far, the hammer begins to bounce, and the pile begins to shiver and spring near the ground, it is time to stop driving, unless the pile is disproportionately long for its diameter. In fairly homogeneous ground, if the driving becomes hard and the hammer starts to

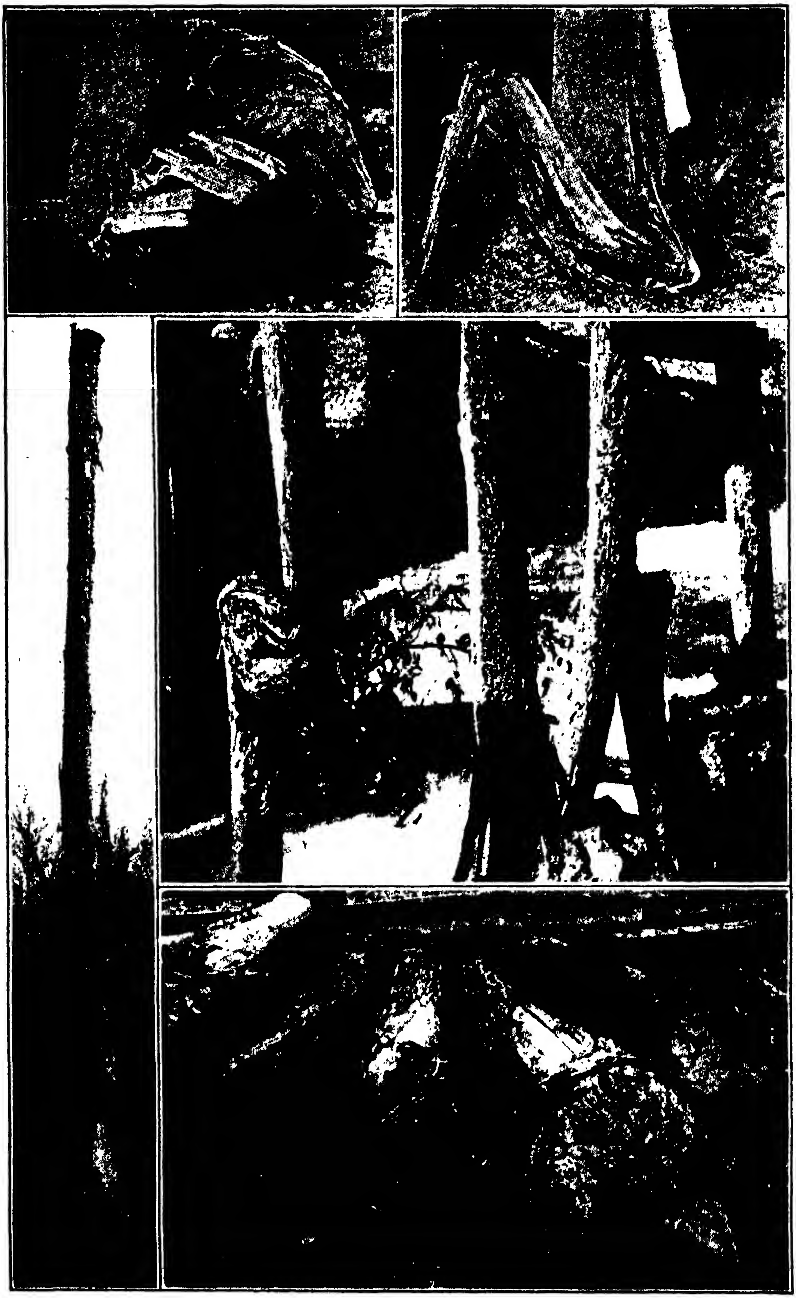


FIG. 4-9a.—Examples of Overdriven Piles Exposed by Subsequent Excavation.

bounce, it is usually wise to stop driving. If driving is continued and the rate of penetration is irregular, it is probably safe to assume that the pile is either brooming at the tip or fracturing at some intermediate portion of its length. If a pile suddenly changes direction, there is but little doubt that it has broken. In general, when a pile that has been sinking easily suddenly stops and the hammer commences to bounce, the driving should cease, as it is probable that the pile has struck a boulder or some other obstruction. The quality of a pile can usually be judged by the behavior of its head under moderate driving. As the driving progresses, the condition of the head also gives some indication regarding the action of the pile below the surface.

4-10. Prevention of Overdriving. The best measures to prevent injury to piles by overdriving include the use of a cap to protect and guide the pile head; the substitution of the steam-hammer for the drop-hammer; the use of the water-jet whenever practicable; and an adequate exploration of the ground to be penetrated. The steam-hammer is more effective than the drop-hammer in securing the penetration of a pile without injury, because of the shorter interval between blows. Some piles have taken over 1,200 blows without any sign of the head being broomed. The use of the water-jet is one of the most effective means to avoid the danger of overdriving, since it reduces the resistance to penetration. Preliminary exploration of subsurface conditions is necessary to interpret properly the behavior of a pile while being driven, as well as to determine the proper length of pile and whether it is to act as a column, or is to support its load by skin friction. If a hard crust has to be penetrated and the piles cannot do so without injury, it is better to use dynamite to break it up, placing it by means of a pipe. Or, a double-strength wrought-iron pipe with a steel point and cap may be driven until it penetrates the hard stratum, and after withdrawing it, the timber pile may be inserted in the hole and driven to the proper depth. This method has been used to penetrate a hard embankment under railroad tracks. In other cases the hard overlying stratum may be removed by dredging or some other method and then replacing the material if it is needed to provide lateral resistance. It is often remarked that judgment based on experience will dictate when to stop driving. The training of the judgment depends not so much on the amount of experience as upon the habit of careful reflection on the results of observations in pile driving and of the probable causes in each case. It is most earnestly to be hoped that the time will come soon when it cannot

be truthfully said that "the most prevalent bad practice in pile driving is overdriving."

4-11. Cutting Off and Removing Piles. When the heads of piles are to be embedded in a footing of concrete, it is unnecessary to have them cut off at exactly the same level; in fact, it is often specified that a certain proportion of them shall be cut off at a higher level than the rest.

When timber caps or timber grillage, however, are to transfer the load from the substructure to the piles, it is important to cut off the piles at the elevations marked on the plans and to specify that concave, convex, or inclined heads will not be accepted. In the open air the cut can be made by an ordinary crosscut saw using two straightedge guides attached to the piles. Another method is to use a circular saw mounted on a vertical shaft that is rigidly supported by a movable frame (Fig. 4-11a).

FIG. 4-11a.—Saw Mounted on a Vertical Shaft.

The piles should be cut at such an elevation that the top of the timber grillage is below the ground water at its lowest stage. Changes in this level caused by probable changes in the drainage system should receive due consideration. Where tidewater has access to piles, it is often customary to keep the timber of the foundation below half tide rather than below low tide, since it will be kept wet continuously by the rise and fall of the tide. The same conditions hold with respect to the heads of piles embedded in concrete, otherwise they will suffer from dry rot.

At the Cambridge bridge over the Charles River the contractors used a heavily constructed machine to cut off foundation piles from 15 to 34 ft. below the water surface. The scow supported regular pile-driver leads 60 ft. high. The saw was 42 in. in diameter and attached to a 4-in. hollow shaft, the bearings of which were supported by a spud or vertical timber 14 in. square which could be easily raised or lowered between the leads. The driving pulley occupied a fixed position and engaged a continuous spline or key attached to the shaft. The saw was operated at a speed of 400 to

500 r.p.m. by means of a 40-hp. engine and a boiler of still larger capacity. The usual rate of cutting 10-in. spruce piles was 600 to 800 per day of 10 hr., with a maximum of 600 in a half day. Horizontal range sights were established and lines painted on the spud to determine the proper elevation of the saw as the tide changed. This method has been used in cutting off piles 60 ft. below water surface.

When timber piles have to be cut off below the water surface to a given elevation, special care is necessary as well as properly designed equipment. Figure 4-11*b* illustrates the machine used in

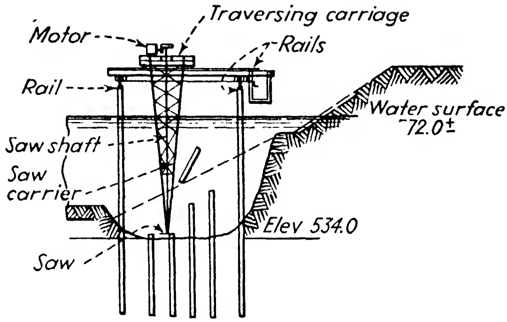


FIG. 4-11*b*.—Cutting off Piles.

cutting off piles in bridgework connected with building the Welland Ship Canal, where the water level was constantly fluctuating. Two rows of piling were first driven on 5-ft. centers, one on either side of the pier site and parallel to its length.

The pile rows were capped with 12-in. timbers carrying a 40-lb. rail. A timber carriage with double-flanged wheels ran on the rails of the pile bents. This carriage, in turn, was similarly fitted on top with a pair of rails or stringers located transversely to the length of the pier. Traveling on the upper track was a second carriage to which was attached the upper end of a structural steel frame of triangular section, 48 ft. long and hanging down into the water. Attached to the one vertical face of the frame, by a series of bearings, was a $3\frac{1}{2}$-in. steel shaft suspended by a thrust bearing carried by the deck of the upper carriage. To the lower end of this shaft a 52-in. circular saw was mounted on a horizontal plane, and on the upper end, above the thrust bearing, was attached a 23-in. crown pulley belt driven by a 75-hp. electric motor on the upper carriage deck.[1]

The cost of cutting off piles by this equipment is given in Art. 4-13.

Where only a small number of piles have to be cut off or in locations where equipment like that previously described cannot be

[1] *Proc. A.S.C.E.*, vol. 58, p. 368, March, 1932.

used on account of interference with structures, a simple device may be adopted and operated by hand. A rigging used on the Chicago and Northwestern Railway consists of a triangular frame in which a saw 4 ft. long is held between the ends of a bent iron bar 2 by ½ in. in section, which forms the other two sides of the triangle, each 8 ft. long. The frame is suspended at its apex from a stick fastened to the lower surface of the stringers of a pile trestle and operated by hand near the water surface. Another device consists of a vertical frame formed by two timbers crossing each other like the letter X; at the crossing a pin is driven into the pile to be cut off; the saw is held between the lower ends of these timbers, and the upper ends are braced by a horizontal timber bolted on just below the handles by means of which it is operated.

Piles are often used for temporary construction, such as to support falsework for the erection of a bridge, and have to be removed afterward. If its penetration is not too large, a pile may be pulled by the pile line of a pile driver or with the aid of block and tackle. To reduce the initial resistance, the pile should be tapped by the pile-hammer before pulling it; or, if the water-jet equipment is available, it may be used to loosen the pile so that it can be easily removed. In tide water, piles are sometimes fastened by a chain to a scow at low tide and thus pulled by a rising tide. If hard ground surrounds a pile, it may be started by securely spiking a block of wood on each side and lifting it by the aid of two screw jacks. The methods in common use for removing steel sheet piling (see Art. 7-16) are also applicable to the removal of timber piles.

To remove the falsework piles of the Municipal bridge at St. Louis, a heavy timber tower formed of two bents battered in both directions and thoroughly braced was constructed on two barges.

The barges were placed about 10 ft. apart in the clear so as to straddle the double line of piles forming a bent. Two sets of falls, composed of four-sheave steel blocks, reeved with wire rope, were used and attached to the pile by means of chains. After the pile was lifted about 20 ft. by the main falls, they were disconnected and the pile lifted clear by means of a runner passing through a snatch block attached to the lower chord of the bridge span. From 30 to 45 piles per day of 9 hr. were pulled with this rig.[1]

Another method of removing piles as an obstruction in a waterway is to cut them off with dynamite. A hole about 1½ in. in diameter is bored down along the axis of the pile with a ship auger and one or more sticks of dynamite inserted and exploded. In

[1] *Eng. News*, vol. 67, p. 239, Feb. 8, 1912.

some cases 75 per cent dynamite has made a clean cut. In one instance where a foreman was instructed to use 70 per cent dynamite, he used 40 per cent dynamite, as that was more easily obtainable. As a result the piles were merely shattered and not cut off. Later, on using two $\frac{1}{2}$-lb. sticks of 70 per cent and one of 40 per cent dynamite the largest timber pile was cut off and the top hurled over 50 ft. into the air. The holes bored were $4\frac{1}{2}$ ft. deep, and the cost was 55 cts. per pile for labor, dynamite, fuse, and cap. Dry sand may be used to fill the hole, after inserting the dynamite, but does not need to be tamped.

In a report on the removal of a temporary pile bridge to clear the river for floating ice and logs in the spring, 40 per cent dynamite was stated to be effective. A ring was formed of telegraph wire which was large enough to slip over a pile, three half sticks of dynamite were fastened equidistant on the wire with a percussion cap in each. A fuse was attached long enough to reach the bottom of the river when the wire was dropped over the pile, and this fuse was connected to a battery. All the piles were cut off clean at the bottom, making this method the cheapest and quickest way to remove the obstructing piles.

To remove a cluster of 13 large pine piles at Leavenworth, Kan., which had been sunk by a water-jet and could not be pulled on account of high water and floating ice, a 3-gal. jug was placed in hot water and partly filled with hot sand so as to store as much heat as possible. The remaining space was filled with 30 lb. of dynamite. After arranging an exploder and battery, the jug was corked and lowered through a small opening in the center of the cluster, and on reaching the sand bottom at a depth of 14 ft., it was exploded, thus cutting off the piles at the level of the jug.

4-12. Pile Records and Performances. The number of timber piles that can be driven by one pile-driver gang in a day depends on many factors, such as the size of the pile, the depth to which it is driven, the nature of the ground penetrated, the amount of moisture in the soil, the kind of hammer used, whether a steam- or a drop-hammer, the relation of the weight of the hammer to the weight of the pile, whether a cap or a ring is used to protect the pile, whether or not a water-jet is employed, the character and condition of the pile driver and its equipment, and the training and experience of the crew. Frequently too much time is lost on those operations which do not involve the action of the hammer, like moving the pile driver from one position to another, getting the pile to the driver, and placing it in the leads.

A pile-driving record was established in 1918 when a 12-man crew at Hog Island drove 220 piles averaging 65 ft. in length in 9 hr. and 55 min. The equipment consisted of a Vulcan No. 1 hammer and a skidding-rolling machine with a three-drum 9- by 10-in. hoisting engine, both hoist and hammer being driven by compressed air. On the foundations of the Cambridge bridge over the Charles River built in 1901, the average day's work for a crew was considerably over 100 piles per day of 10 hr., while the highest number driven in a single day of 9 hr. was 212. The piles were about 40 ft. long, and the heads were driven with the aid of a follower to an elevation of 18 ft. below low water. The material penetrated was a hard clay below the upper stratum of softer material. In placing 18,000 piles through 30 ft. of dense coarse sand and clay for the foundations of a power plant in New York City, 50 piles were driven per 8-hr. day per hammer.

In driving piles at the Mare Island Dry Dock 2, the average number of piles driven per shift of 8 hr. was 35 for a period of 3 months, 74 representing the best day's work. The timber piles ranged from 40 to 65 ft. in length, and the penetration varied from 12 to 46 ft. The piles were driven by a heavy steam-hammer with the aid of a 40-ft. follower to such an elevation that they could be cut off at 36 ft. below low water. The piles were located on intersecting lines with unusual accuracy, the pile driver being fitted with sliding extension leads 88 ft. long. In driving 2,650 wood piles 55 ft. long through typical Chicago clay in 1929 for a building foundation, the average rate was 37 piles per 8-hr. day per hammer. Seven per cent of the time was spent in moving runs and timbers for driving, 3 per cent in moving pile butts and debris and 1 per cent was lost through breakdowns.

Forms of pile records differ more or less according to the character of the structure. The record form for railroad trestles adopted by the American Railway Engineering Association is published in the Manual of that association. The four horizontal lines above the tabular form are for the bridge number or name, its location, the weight and kind of hammer, the date, and a statement that bents are numbered from the north or east end and that piles are numbered from left to right. The vertical column headings are as follows: Date; Bent Number; Number of Pile; Size of Pile, including, Tip End, Butt End and Length; Kind of Wood; Length of Cutoff; Distance from Base of Rail to the Ground; Total Penetration; Average for the Last Five Blows, of the Drop of the Hammer and of the Penetration; Kind of Soil; and Remarks.

In foundations of buildings, bridge piers, wharves, etc., the piles are usually arranged so that the rows in one direction can be lettered *A*, *B*, *C*, etc., while the piles in each row are numbered. Any pile can thus be designated by a letter combined with a number, thus: *E*19. In the foundations of pivot piers and of circular buildings the piles are preferably arranged in circular rows, which may be similarly designated by letters.

4-13. Pile Costs. The cost of a pile in place is made up of two parts, (*a*) the cost of the pile itself and (*b*) the cost of driving. The cost of the pile will depend on the length of the pile, the grade, the location where driven, and whether plain or treated. For example, where the price is 20 cts. per foot for a plain pile in lengths up to 50 ft., the price per foot for an 80-ft. pile will be about 45 cts. per foot. Piles of extreme lengths are much higher. For example the 135-ft. piles illustrated in Fig. 3-4*a* cost about $1.10 per foot, one-half of this representing freight charges.

The cost of pile driving will vary between extremely wide limits and will depend on many factors such as (*a*) kind of material penetrated, (*b*) number of piles to be driven, (*c*) whether or not shoes are needed, (*d*) whether or not placed through water, (*e*) whether driven vertically or on a batter, (*f*) whether or not a water-jet is required, etc.

The total field cost of piling, per linear foot, for bridge structures in connection with the Welland Ship Canal, the work being done between 1924 and 1930 and the piles varying in length from 30 to 60 ft., was as follows:

Piles	$0.3100
Peeling, pointing, and rafting	0.0233
Driving	0.0480
Repairs and general expense	0 0372
Total field cost per foot	$0.4185

On 13 bridge jobs in West Virginia from 1930 to 1938 involving 54,000 lin. ft. of plain timber piles the contract prices per linear foot varied from 35 to 80 cts., with an average of 47 cts. On five jobs involving 26,000 lin. ft. of creosoted piles, the minimum, average, and maximum contract prices per foot were 75 cts., 84 cts., and $1, respectively. These figures include both furnishing and driving the piles.

In the construction of the viaduct approaches of the Southern Railway over Chattahoochee River completed in 1907, the cost of the piles per linear foot under the viaduct footings was found to be as follows, there being 13,750 lin. ft.:

Piles	$0.277
Shoes, spikes, and rings	0.026
Coal, oil, waste, rent of driver, etc	0.055
Labor for pile driving	0.232
Labor for sharpening piles	0.006
Total cost per linear foot of piles	$0.596

The cost of freight and of train service is included in the item for materials. One row of pedestals came beneath the old trestle, and this required considerable manipulation of the driver and loss of time in working around the bents, which made the cost of pile driving rather high. The actual labor of driving piles for the outside row of pedestals was only a little over 9 cts. per linear foot of piles in the leads.

The method used for cutting off the pile tops at the Welland Ship Canal work is described in Art. 4-11. The cutting-off cost per pile was as follows:

Erection of cutoff outfit at pier site	$1.064
Operating cutoff outfit	2.159
Dismantling and removing forms	0.396
Pile cutoff outfit, first cost	2.353
Total cost of cutting off one pile	$5.972

4-14. Deterioration of Timber Piles. Deterioration of timber piles is due to three agencies, (*a*) decay, (*b*) insects (termites and wharf borers), and (*c*) marine borers. The first two attack above water level and the third below water level. Marine borers are much more destructive to piling than decay or insects.

Decay is caused by the presence of low forms of plant life called "fungi," which subsist on certain substances in the wood. The rate of disintegration of the wood depends on the moisture and temperature conditions, as well as on air conditions. Some species of fungi develop most rapidly under high moisture conditions and others with less moisture. High temperatures usually result in accelerated action. Timber that is continuously below water level does not decay.

Termites, which somewhat resemble ants in appearance, are of two types, the "subterranean" and the "dry wood." The subterranean termite requires moisture and so must have access to the ground at all times. This type is widely distributed throughout the United States and does a great deal of damage to wood structures. The dry-wood termite, which flies and does not need contact with the ground, is not found much north of Norfolk, Va., on the East coast and San Francisco, Calif., on the West coast. The

chief food of termites is cellulose. There are few or no species of wood that are immune to the attack of termites, although the heartwood of some trees native to the Orient are resistant. Redwood, heart cypress, and heart longleaf pine, containing a large amount of pitch, are more resistant than other native woods.

The damage by wharf borers is done by the young of a winged beetle. The beetle lays its eggs in the cracks of the timber, and the larvae or worms that destroy the wood are hatched from these eggs. Although they do not work below water level, they seem to prefer locations not too far above high water or timber that is wet by salt spray at times.

4-15. Marine Borers. Marine borers destructive to timber belong to two families, the Crustacea (related to the crab and lobster) and the Mollusca (related to the clam and oyster). The three genera of the Crustacea of economic importance are *Limnoria*, *Sphaeroma*, and *Chelura*. *Limnoria*, of which there are many species, resemble the ordinary wood louse, having bodies from $\frac{1}{8}$ to $\frac{1}{4}$ in. long and about one-third as wide. Like the wood louse they belong to the order of Crustacea known as Isopoda. They attack timber by running burrows, about $\frac{1}{2}$ in. long and $\frac{1}{20}$ in. in diameter, just beneath the surface of the timber, the wood serving both as a food and as a shelter. They generally follow the softer spring wood between the harder layers of autumn growth, their burrows being so numerous that the surface layers of timbers are rapidly destroyed (see Fig. 4-15*a*). In creosoted wood they frequently enter at some point of thin treatment, such as a knot or point of abrasion, and work in the untreated center of the stick.

FIG. 4-15*a*.—Action of Limnoria and Chelura in Boston Harbor. (*Courtesy of Civil Eng.*)

Limnoria attack piles mostly between an elevation slightly below low tide and about half tide. However, the attack may be heavy down to mud line, and they are sometimes found above normal high tide. This borer is widely distributed along the American coasts, extending from Newfoundland to the Falkland Islands and from Alaska southward. Unlike the *Teredo* which lives all its life in the particular hole which it starts, the *Limnoria* can and does change its abode quite frequently. Studies made at

Beaufort, N. C., indicated that *Limnoria*, although active, do not breed there when the temperature is below 14°C. On the other hand, in San Francisco they breed all year although the temperature is much below 14°C.

The limits of necessary salinity of water for *Limnoria* have not been definitely determined, but some tests have shown that a salinity as low as 6.5 parts per 1,000 is lethal within 24 hr., while a salinity of 10 parts per 1,000 is probably ultimately lethal. A salinity as low as 16 parts per 1,000 results in a decided decrease in their activity, whereas a salinity of 30 parts per 1,000 may be considered ideal for their development.

The crustacean borers of the genus *Sphaeroma* have the same general structure as *Limnoria* but are much larger, sometimes reaching a length of $\frac{1}{2}$ in. and a width of $\frac{1}{4}$ in. In general *Sphaeroma* are not of much economic importance, although one or two species have apparently done considerable damage. They bore in wood apparently only for the purpose of obtaining shelter and are found in water where the salinities range from that of normal sea water down to that of water that is only faintly brackish. Creosoted timber is not immune from attacks by this borer.

Chelura belong to the order of Crustacea of the amphipod genus and differ to a considerable extent from *Limnoria* and *Sphaeroma*, their bodies being more attenuated and much more prominently equipped with appendages, which give them a lobsterlike appearance. *Chelura* are usually found with *Limnoria* in marine structures and are rarely to be seen in the absence of the latter. Where present they are even more destructive than *Limnoria*, but until recent years they were rarely found in American waters. The first recorded examples of pronounced activity of *Chelura* in the United States was at Boston in 1934.

4-16. Mollusca. Mollusca are bivalves distantly related to the clam family, the most important genera in America destructive to timber being the *Teredo*, *Bankia*, and *Martesia*. The last-named resembles the clam in general appearance, being wholly enclosed within a shell, but *Teredo* and *Bankia* have wormlike shapes, with heads of shell material.

Mollusca enter timber when very small and spend there their entire lives, burrowing and enlarging their holes as they grow. A pile may have only a few pinlike holes exposed and yet be completely honey-combed on the inside. *Teredo* are generally only a few inches long, although specimens are sometimes found 30 in. long, while some species of *Bankia* reach a length of 4 ft. and more

and a diameter of 1 in. Borings made by *Martesia* are generally not over 1 in. in diameter and $2\frac{1}{2}$ in. in length. A large part of the food of Mollusca probably consists of microscopic plants and animals in the water, but evidence also indicates that wood fragments of the burrows are also utilized as food.

Shipworms of the *Teredo* genus are found in all parts of the world. They are very active along the coasts of this country and are the most destructive of all marine borers in New England waters (Fig. 4-16*a*). Their rate of growth depends on several factors, such as salinity of the water, temperature of the water, and the absence of pollution. No general statement can be made as to the effect

FIG. 4-16*a*.—Action of Teredo Navalis at New Haven, Conn. (*Courtesy of Civil Eng.*)

of water salinity, since all species of the *Teredo*, of which there are about 300, do not exhibit the same characteristics. Furthermore, it appears that the total salt content may be of less importance than the relative percentages of the seven or eight more important salts found in normal salt water. *Teredo navalis*, one of the most destructive types, thrives best at salinities between 15 and 30 parts per 1,000 but is very destructive with salinities as low as 10 parts per 1,000. It will die when the salinity falls below about 5 parts per 1,000.

Little is definitely known regarding the effect of temperature, but, in general, marine-borer activity increases with a rise of temperature. Studies indicate that on the New England coast north of Cape Cod *Teredo navalis* breed only in July and August; south of this the breeding season is longer. In San Francisco Bay they breed from July to December, occasionally continuing into January. As boring is done only during the breeding season, a knowledge of breeding habits becomes most important.

For many years certain harbors which are known to be highly polluted with sewage and trade wastes have been remarkably free

from the ravages of marine borers. In a number of places, particularly along the New England coast, a reduction of pollution has resulted in the appearance of *Teredo* and *Limnoria* in excessively destructive numbers.

Bankia, although not so widely distributed as *Teredo*, may be even more destructive. In lightly infected piling the burrows of *Bankia* usually penetrate more deeply before turning to run parallel with the grain than do the burrows of *Teredo*. Both types tend to concentrate near the mud line. *Bankia* occur most abundantly from New Jersey south to Florida, along the Gulf of Mexico, and from San Francisco north on the Pacific coast. Bankia in general require higher salinities than *Teredo*, or about the same as *Limnoria*.

Martesia is not, properly speaking, a shipworm, but it is a wood-boring mollusk capable of doing considerable damage. Reports indicate that *Martesia* bore even in creosoted wood.

4-17. Life of Untreated Piles. Where the waters are infested by marine borers, timber piles which are not chemically treated or mechanically protected have a very short period of usefulness. The average life of an untreated timber pile on the coasts of the South Atlantic, Gulf, and Pacific states ranges from about 8 months to 2 years. The development and activity of the borers are stimulated by high temperatures, and hence in some of the more northern coasts the average life may extend to 2 or 3 years. The minimum life of service is considerably shorter. In very salty water and during a hot season a pile 18 in. in diameter may be entirely honeycombed in 3 months. In the summer of 1924 some piles in San Francisco Bay were destroyed in 2½ months. It is claimed that in the vicinity of Puget Sound, a stick of timber, rough sawed, will last about 8 months, a peeled pile will last 1 year, and a pile with the bark on will last 1½ years.

Probably no timbers of the temperate zone are immune from borer attack, although some are less subject to attack than others, but some tropical and foreign woods seem to be quite resistant. Some of the species more or less immune are cottonwood, palmettos, mangroves and palms, Eucalyptus, Philippine woods and greenheart. Among the woods that have shown a pronounced resistance to *Teredo* in the Canal Zone are manbarklak, angelique, sponsehoedoe, foengoe, anoura, turpentine wood, malabayabas, kajol Lara, alcornoque, and kajol malas.

Cottonwood has given good service under some conditions, but not so good under others. Palmettos, mangroves, and palms give better records than northern timbers. The foregoing timbers, on

account of their lack of strength, are not considered as structural timbers and so their use must be limited to light structures. Manbarklak has given the best of service in the Canal Zone. The trees grow about 100 ft. tall and 3 or 4 ft. in diameter and weigh about 76 lb. per cu. ft. Both manbarklak and angelique contain particles of silica which are said to prevent entrance of borers.

4-18. Chemical Preservation. Numerous chemicals have been employed to impregnate timber. Such chemical compounds as zinc chloride, copper sulphate, and mercuric chloride, which are extremely poisonous to animal life, have been used in the preservation of wood, but they tend to become leached out by the action of sea water.

Creosote impregnation has proved one of the most satisfactory ways of protecting piles. The durability of creosoted timber piles depends on several factors. The timber must be of good quality, must be free from decay, and should have sufficient sapwood to take the requisite amount of creosote oil. For marine structures this should be about 22 lb. of creosote oil per cubic foot of timber if the wood can be made to absorb this amount. For piles used on land above ground-water level the treatment should consist of 12 lb. of creosote oil per cubic foot of timber when the thickness of sapwood is less than 2 in. and 16 lb. of creosote oil per cubic foot of timber when the thickness of sapwood is 2 in. or more. The creosote oil must be of high grade, with the proper chemical constituents and physical qualities, and the seasoning and chemical treatment have to be conducted so as to secure the desired penetration.

The piles should preferably be air-seasoned before treatment, for this materially reduces the treating cycle and a better treatment is secured. However, artificial seasoning is often used. For southern pine this consists of a steaming and vacuum process. After placing the timber in a large cylinder, the doors are closed and steam admitted under a pressure of 20 to 25 lb. per sq. in. As soon as the sap is entirely liquefied, which usually does not exceed 20 hr., the steam pressure is released and both air and the remaining steam are exhausted by creating a vacuum of not less than 22 in. Douglas fir, a wood difficult to treat, may be artificially seasoned by boiling in oil under a vacuum.

On completion of the seasoning treatment, without breaking the vacuum, the cylinder is filled with creosote oil heated to a temperature between 170 and 200°F. The pressure is then gradually raised to the desired amount, at least 100 lb. per sq. in., and maintained until the timber has received the required amount of oil. Where

the wood has been air-seasoned, the process starts with the establishment of the vacuum. On completion of the penetration treatment, the oil is drawn out of the cylinder and a vacuum maintained until the material can be removed free of dripping preservative.

When the treatment is carefully done and the full amount of impregnation with the best quality of creosote is secured, the timber will not be materially reduced in strength and it will be protected effectively from most marine borers for an average period of from 15 to 30 years. Experience shows that, where borers have attacked creosoted piling, this attack always starts where the penetration of the oil is small or where the surface of the pile has been damaged, hence the necessity of full and uniform creosoting treatment as well as careful handling of piles. A survey of the whole San Francisco water front revealed that 80 per cent of all holes through which borer entrance had been gained were caused by "dogging" the piles and not subsequently plugging the holes.

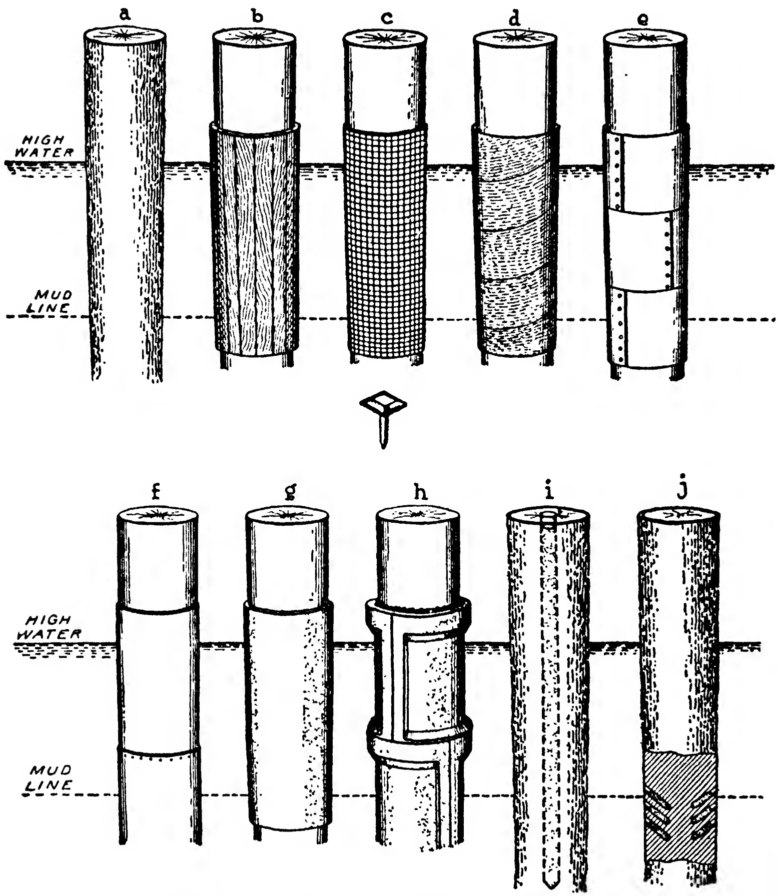
The American Railway Engineering Association specifies:

> Treated piles shall be handled with rope slings, taking care to avoid dropping, bruising or breaking of outer fibers, or penetrating the surface with tools. Sharp-pointed tools shall not be used in handling treated piles or turning them in the leads.
>
> The surface of treated piles below cutoff elevation shall not be disturbed by boring holes or driving nails or spikes into them to support temporary material or staging.

In some piling work in Portland, Maine, in 1922, where southern pine piles from 85 to 89 ft. long were used, the bark was left on for the lower 30 ft., that part which would be in the ground after the piles were in place. It was thought that the presence of the bark would largely prevent the entrance of creosote oil where it was not needed.

The limitations of space prevent the insertion of specifications for the chemical preservation of piles, as well as a discussion of the various methods of treatment and of tests to be applied to the oil, etc.; hence the reader is referred to the *Reports* of the Committee on Wood Preservation of the American Railway Engineering Association and to the *Proceedings of the American Wood Preservers' Association.* Valuable information on the general subject of marine borers will be found in "Marine Piling Investigations," published (1924) by National Research Council, and "Marine Borers and Their Relation to Marine Construction on the Pacific Coast," published (1927) by the San Francisco Bay Marine Piling Committee and sold by the University of California Press.

4-19. Mechanical Protection. As the attack of marine borers does not extend far below mud line nor above high-water level and as chemical treatment must be applied along the entire length of the pile, unless spliced piles are used, mechanical protection may be more economical than chemical protection. Mechanical protection



FIGS. 4-19*a* to *j*.—Mechanical Protection of Timber Piles.

may also be necessary after chemically treated piles have lost their immunity to borer activity due to leaching out of the chemicals. Many methods for mechanical protection have been developed, some worthless and others quite effective. Some of these methods are illustrated in Figs. 4-19*a*–*j*, which are reproduced from *Circular* 128 of the United States Forest Service, entitled Preservation of Piling against Marine Borers, by C. Stowell Smith.

Figure 4-19*a* illustrates a pile with the bark left on. Protection is afforded by this means as long as the bark remains absolutely intact, probably due to the presence of certain oils and acids in the bark. Experience shows that the life of the pile may be prolonged from 1 or 2 years to many years. In Boston where *Limnoria* have been very active in the last few years, piles driven in 1913 were in very good condition in 1934 only in those cases where the original bark remained. On the other hand in San Francisco specimens of bark have sometimes shown large burrows of *Teredo*. This type of protection must be considered as highly problematical because of the facts that (*a*) all borers are apparently not repelled and (*b*) it is very difficult to maintain a perfect bark covering.

In Fig. 4-19*b* the pile is sheathed with planks, a method formerly thought to be effective but now recognized to be one of but slight value. If tar paper, processed felt, or other similar material is used under the sheathing, or if the sheathing is creosoted, this method may be fairly effective.

In ancient times attempts were made to protect galleys by driving the hulls full of nails, and this method has been applied to piles as shown in Fig. 4-19*c*. The nails are sometimes driven so that the heads cover only a portion of the surface—perhaps 50 per cent—the thought being that the rust formed on the surface of the pile between nail heads would discourage borer action. The Santa Fé Railroad used this method in 1881, using 3*d* and 4*d* nails spaced ½ in. centers. This method at the present time is only of historical importance, for, though it is fairly effective, it is quite expensive. A method somewhat similar to the use of nails but less effective is the application of strips of sheet iron with open spaces between the strips. Experience has shown that the presence of rust will not stop borer action.

Figure 4-19*d* illustrates a type of protection where the pile is covered with burlap, felt, or similar material, soaked in various chemicals. In some cases the surface of the pile is painted prior to the placing of the burlap. The chief advantage of this type of covering is its relative cheapness and ease of application. The main disadvantage is the difficulty of keeping the protective coating intact, both during the placing of the piles and after they are in service. Where the work has been well done in locations protected against serious storm action, experience indicates that the piles will be ensured against borer action for an average period of between 5 and 8 years and sometimes longer.

Figures 4-19*e* and *f* illustrate the use of a metallic sheathing consisting of thin sheets of iron, zinc, yellow metal (an alloy of copper and zinc), or copper. Iron is seldom used as it corrodes rapidly. Zinc and yellow metal corrode rather rapidly and are expensive. Copper sheathing is expensive, but its rate of corrosion is much less than that of the other metals noted. However, it is soft and easily torn and abraded, and because of its high junk value it is likely to be stolen. Where used under proper conditions, records indicate that it may give protection for 20 years or more.

Concrete casings, as exemplified in Fig. 4-19*g*, have been quite widely used and with success where a good grade of concrete or mortar carried well below scour line was employed. These casings may be precast or cast in place. Precast casings may be placed around the pile either before or after the pile has been driven. The Ripley combination pile (Art. 6-14) is one example of a casing placed before driving the pile; another example is the pile covered with Shotcrete (Art. 6-14), a mortar placed pneumatically with a cement gun.

Where precast casings are placed after the pile has been driven, the reinforced-concrete protective shell is cast, on land, entirely separate from the pile and then slipped over or around the pile after the latter is in place. These shells are usually used full-length, although sectional jackets have been designed with special details for connecting the sections as they are lowered into position. The casings are usually round with a minimum thickness of shell of $2\frac{1}{2}$ to 3 in. The diameter is enough greater than that of the pile to allow for irregularities in the latter and also for the insertion of a pipe between the pile and the shell with which to pump out and grout the space between the jacket and the pile.

In building pier 50 in San Francisco in 1925 the jackets had a minimum outside dimension of 20 in. and a minimum thickness of 3 in., and were circular on the inside and octagonal on the outside. The reinforcement consisted of eight $\frac{1}{2}$-in. square deformed bars spirally wrapped with No. 6 wire on a 4-in. pitch. The concrete consisted of one part of cement to $4\frac{1}{2}$ parts of graded aggregate. As the jacket was lowered into position over the pile, the mud was forced out from between the pile and jacket by means of a gasket made of old rubber hose. Later the open space was sealed by grouting through the water. After the grout had set, the space between the jacket and pile was pumped out and filled with concrete.

Sectional jackets may consist of reinforced concrete or of vitrified clay pipe (Fig. 4-19*h*) slipped over the heads of the pile, the intervening space being filled with concrete or sometimes with sand. To protect piles in this manner after the caps or superstructures are in place requires the use of pipe divided longitudinally into halves. Figure 4-19*k* gives the dimensioned plans of a concrete pipe of this kind known as "lock-joint pipe." The two halves are placed around the pile and locked together by inserting wooden keys, soaked in hot tar, on the scarf joints. Reinforced-concrete pipe is superior to vitrified-tile pipe on account of its greater resist-

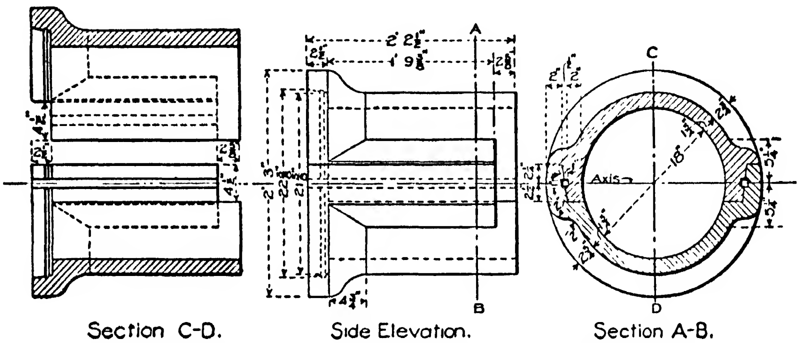


FIG. 4-19*k*.—Sectional Concrete Casing for Timber Piles.

ance to accidental or other blows. In unusually exposed situations cast-iron pipe has been used.

There are several methods of placing cast-in-place casings. One form of construction consists of slipping a corrugated iron culvert pipe over the end of the driven pile, forcing the pipe into the sand or mud, and filling the space between the form and pile with concrete. The corrugated pipe is left in place. Another method uses forms of sheet steel in sections 18 in. long and split longitudinally into halves. Vertical angles are riveted to these halves in such a way they may be clamped together with rubber gaskets between them to make tight joints. The lower end of each section is slightly reduced in diameter and fits tightly into the slightly enlarged upper end of the next lower section.

Figures 4-19*i* and 4-19*j* illustrate the protection of piles by boring holes into them and filling the holes with a poisonous substance. This treatment has been discontinued.

CHAPTER V

BEARING POWER OF PILES

5-1. General Considerations and Load Tests. The safe load on a pile may depend on (*a*) the strength of the timber acting as a column, (*b*) the frictional resistance between pile and earth plus point bearing, or (*c*) the bearing capacity of the soil on horizontal planes below the foot of the piling. Simple structural analysis, as explained in the next article, will be sufficient for determining the size and number of piles where the first condition obtains.

Where the second condition obtains, a static test may be applied to a single pile or to a small group. Such a test is most easily made by building a platform on the pile to be tested and loading the same

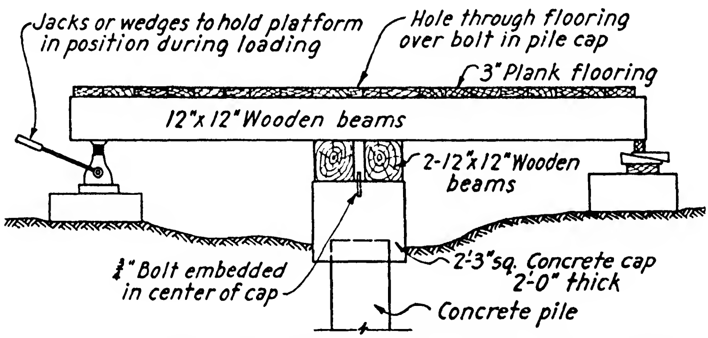


FIG. 5-1*a*.—Apparatus for Testing Load-carrying Capacity of Piles.

with pig iron or other heavy material. Other loading methods are described in Art. 6-19. Single piles have been tested by applying the load through a hydraulic jack reacting against an I-beam fastened by steel straps with tabled fishplate joints to two anchor piles. The results of this kind of test must be used with great caution, for the stress conditions in the soil are quite different from those obtaining in an actual foundation bed. For example, the upward forces in the two anchor piles equal the downward force on the test pile; hence the stress in the soil below the piles is zero. Figure 5-1*a* illustrates a type of platform recommended by the Portland Cement Association[1] for testing concrete piles (see also

[1] See bulletin entitled "Concrete Piles," Portland Cement Association.

Art. 6-19), which is quite suitable for timber piles. In running the test, the load should be applied in increments of 5,000 to 10,000 lb., with sufficient time after each increment to allow for full settlement. The movement of the top of the pile will be measured for each load application, after which a graph may be plotted similar to that shown in Fig. 5-1*b*. The measured movement consists of the elastic deformation of the pile plus its permanent settlement. If a hydraulic jack is used, the load can be released after each application and the permanent settlement found by subtracting the recovery from the total movement.

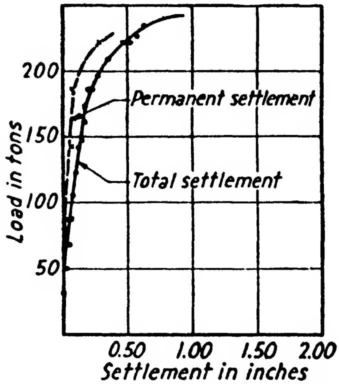


Fig. 5-1*b*.—Load Settlement Diagrams for a Concrete Pile.

For some types of soils the load settlement curve breaks sharply, in which case the load at the break of the curve may be considered to represent the ultimate capacity of the pile. If there is no break in the curve, an arbitrary limiting value of settlement, such as $\frac{1}{4}$ or $\frac{1}{2}$ in., may be selected. For example, quoting from the Boston Building Code:

a. A pile to be tested shall be loaded to at least twice the proposed working load, the load being applied in increments of not over 10,000 lb. At least 4 hr. shall elapse between the addition of successive increments. Measurements of the settlement, accurate to $\frac{1}{32}$ in., shall be taken and recorded immediately before and after each increment of load is added. In determining the settlement, proper deduction shall be made for elastic compression of the pile under test load.

b. The allowable pile load shall not exceed one-half of that causing a total settlement of $\frac{1}{2}$ in. which remains constant for 48 hr. . . .

The Building Code (1938) of New York City provides:

Tests shall be made with 150 per cent of the proposed load and such tests shall be considered unsatisfactory . . . if after the piles being tested have been standing 24 hr., the total net settlement after deducting the rebound is more than 0.01 in. per ton of total test load.

Where it is not feasible to make load tests, the bearing capacity of a pile may be roughly determined by the application of some pile-driving formula, as discussed in Arts. 5-3 and 5-4.

A pile may be amply strong to take a given load and the frictional resistance of the soil may be adequate to take the same out

of the pile, and yet the soil may not have sufficient supporting capacity to carry this load. In this case the problem becomes one of proper pile spacing (Art. 5-7). Either the piles must be spaced far enough apart so as not to overload the soil or else the design load on each pile must be limited to satisfy the same requirement. In other words, the allowable loading for a group of piles composing a footing must be limited by the provision that the vertical pressures on the bearing materials at or below the points of the piles, produced by the loads on all the piles, must not exceed the allowable bearing values of such materials (Art. 1-15).

The Boston Building Code states:

> Piles or pile groups shall be assumed to transmit their loads to the bearing materials by spreading the load uniformly at an angle of 60 deg. with the horizontal, starting at a polygon circumscribing the piles at the top of the satisfactory bearing stratum in which they are embedded, but the area considered as supporting the load shall not extend beyond the intersection of the 60-deg. planes of adjacent piles or pile groups.

In addition to load limitations on timber piles by reasons noted above, it is quite common for specifications to place an empirical limitation on the same. For example, the New York City Code (1938) specifies:

> The maximum allowable load on a wood pile having a 6-in. point shall be 15 tons; on a pile having a point of 8 in. or over, the maximum allowable load shall be 20.

According to the 1935 specifications of the American State Highway Officials Association:

> In general, piles shall be required to develop a bearing capacity of not less than 15 tons and not more than 25 tons each. However, the character of the soil penetrated, conditions of driving, distribution, size and length of the piles involved, and the computed load per pile shall be given due consideration in determining the reliability of driven piles.

5-2. Piles Acting as Columns. When piles on land project some distance above the surface, they are usually held in position laterally by diagonal bracing whenever there is sufficient room for it so that the pile is not subject to direct bending. An example of this construction occurs in pile trestle bents. If the vertical distance between points of connection for the bracing is large, the pile must be designed to provide for column action.

Piles driven in water are frequently not braced; hence it is essential to design them with regard to their strength as long columns, since this may be the limiting condition rather than the bearing

power of the ground penetrated. If the substructure placed upon the piles is not held laterally except by vertical piles, then the piles act like columns with the upper end practically free and the lower end fixed at some elevation which depends upon the material penetrated. To determine this elevation is the principal problem. Usually it cannot be taken at the bed of the river or lake, since the material there is soft or yielding. Even if the bottom consists of firm gravel and sand, the required elevation is 1 ft. or more below its surface. When the material is mud or silt, grading slowly into more compact material, the upper strata give relatively little resistance to the lateral deflection of the pile. That the resistance of such material is greater than may be naturally inferred from its consistency is proved by the fact that at New York piles frequently break off in case of trouble at approximately the mud line of North River silt.

It has been proposed as a reasonable assumption to consider the lower third of the softer strata, which overlie the hard stratum, to be effective in lateral resistance and to ignore that of the upper two-thirds. This makes the assumed free length of the pile column equal to the distance from the pile cap to the river bottom plus two-thirds of the penetration in distinctly soft ground. The strength of such a column is equivalent to that of a column of double the length with both ends round. It must also be remembered that the strength of a group of pile columns has only the strength of one column multiplied by their number, since there is no provision made to resist their movement longitudinally with respect to one another. In this respect they are analogous to composite posts (see "Timber Design and Construction," Jacoby and Davis, Arts. 52 and 53).

It frequently happens that bearing piles transmit nearly or all of their vertical load to a hard substratum, overlaid by softer material, which will yield laterally under pressure. In all such cases the piles should be designed as columns.

If the foot of a pile bears on rock and the overlying material is not able to resist its lateral displacement, it may be necessary to drill shallow holes into the rock. In shelving rock this method of preventing displacement is of especial importance. Sometimes riprap is used for this purpose, but its uneven weight upon the material overlying the rock has been known to cause sliding with disastrous results.

The section area of the post should be large enough to provide adequate bearing area and the taper of the timber pile should be

as small as possible. In some cases it may be desirable to place the butt at the foot of the pile. The foot should not be pointed unless it is necessary to secure penetration for a short distance into material like hardpan, to prevent its lateral displacement.

The greatest care should be taken in driving piles that are expected to rest in bedrock or to penetrate slightly into hardpan in order to prevent shattering or crushing the feet of piles, otherwise their supporting power may be seriously impaired. When the overlying material is muck or silt, or soft, yielding material, it is preferable to omit the resistance, if any, due to skin friction in designing the pile.

The conditions described in the preceding paragraphs often apply to falsework piles used to erect bridges. In any case it is desirable to brace the piles effectively by means of sway bracing, but in rivers that carry a large amount of driftwood during flood seasons, or large masses of floating ice, it is essential to provide the piles with carefully designed sway bracing and to add lateral bracing. Experience amply justifies special caution in design and construction for this purpose.

The unit stress in the outer fiber which may safely be allowed depends upon the species of wood as for ordinary wooden columns, but some reduction is usually made on account of the piles being water-soaked. Sometimes its value is reduced by the use of a lower grade of timber, which has more knots and other defects than are permitted by specifications for structural timber. When no account is taken of the species of wood, specifications sometimes give the working unit stress in the outer fiber as 600 lb. per sq. in., reduced for pile columns to $600\ (1 - l/60d)$, in which l is the unsupported length in inches and d the diameter at the middle of the unsupported length.

When piles act as columns or are subject to bending moment, especial care should be taken to drive them accurately in position; for, if they have to be forced laterally into line, account must be taken of the initial flexural stress thus produced.

5-3. Rational Pile-driving Formulas. The general characteristics of energy transfer from the hammer to the pile are discussed in Art. 4-1. Using the nomenclature of that article, together with the following, all forces being expressed in pounds and all distances in inches:

Let R_d = ultimate resistance of the pile

R_a = allowable load on the pile

s = penetration of pile per blow

E = modulus of elasticity of pile
A = mean cross-sectional area of pile
L = length of pile
C_1 = elastic compression of pile
C_2 = rebound of pile due to elasticity of soil
C_3 = elastic compression of pile cap
$C = C_1 + C_2 + C_3$

We have the following energy conditions,

$$eW_hHK_1 = eW_hH\frac{r(1+n)^2}{(r+1)^2} = \text{K.E. of pile (see Eq. 4-1}c\text{).}$$

$$eW_hHK_2 = eW_hH\frac{(r-n)^2}{(r+1)^2} = \text{residual K.E. of hammer (see Eq. 4-1}d\text{).}$$

$$\frac{1}{2}R_d\frac{R_dL}{AE} = \frac{1}{2}R_dC_1 = \text{energy lost in elastic compression of pile.}$$

$$\tfrac{1}{2}R_dC_2 = \text{energy lost in elastic compression of soil.}$$

$$\tfrac{1}{2}R_dC_3 = \text{energy lost in elastic compression of pile cap.}$$

$$R_ds = \text{useful work done on pile.}$$

Equating the kinetic energy of the pile to the useful work plus losses, we have

$$eW_hHK_1 = \tfrac{1}{2}R_dC + R_ds,$$

$$R_d = \frac{eW_hH}{s + \frac{C}{2}}K_1, \tag{5-3a}$$

where
$$K_1 = \frac{r(1+n)^2}{(r+1)^2}$$

Where the residual kinetic energy of the hammer is added to the kinetic energy of the pile,

$$eW_hHK_1 + eW_hHK_2 = \tfrac{1}{2}R_dC + R_ds,$$

$$R_d = \frac{eW_hH}{s + \frac{C}{2}}(K_1 + K_2), \tag{5-3b}$$

where
$$K_1 + K_2 = \frac{r + n^2}{r + 1}$$

These formulas were first developed by Hiley. Values of K_1 and K_2 are given in Table 4-1*a*. When $r = n$, in other words $W_h = W_pn$, the values of R_d from the two equations become equal, for in this case there is no residual hammer velocity (see Art. 4-1). Where r is less than n, Eq. 5-3*a* must be used, for under this condition the residual hammer velocity is upward and so its energy

cannot be transferred to the pile. Where r is greater than n, Eq. 5-3b may be used, although it may give results a little high if the residual hammer energy is not completely transferred to the pile.

In using these formulas, the values of e, n, and C must be evaluated. The following values of e are probably about right.

TABLE 5-3a

Type of Hammer	Value of e
Drop-hammer, free fall	1
Drop-hammer, line attached	0.75
Single-acting steam-hammer	0.9
Double-acting steam-hammer	1.0

As stated in Art. 4-3, for a free fall the energy losses in the hammer are small, but with a line attached to the hammer, the energy absorbed in pulling the line against the drive resistance is considerable. Taking the tests given in Art. 4-3 on the relative penetration with free and restrained falls and applying Eq. 5-3a, we note that the values of e are inversely proportional to $s + \frac{C}{2}$. For these tests $C/2$ is small as compared with s; hence roughly e may be said to be inversely proportional to s, hence the values of e for restrained falls will be $0.5/0.7 = 0.71$, $0.7/0.9 = 0.78$, and $0.32/0.4 = 0.8$ for piles 1, 2, and 3, respectively.

The coefficient of restitution, n, may theoretically vary from 0 to 1. The following values are widely used:[1]

TABLE 5-3b

	n
Cast iron on steel	0.55–0.60
Cast iron on concrete	0.40
Cast iron on wood	0.20–0.25

The elastic compression of the pile is given by the formula, $C_1 = R_dL/AE$ which for a timber pile becomes $R_dL/(A \times 1{,}500{,}000) = 0.000{,}000{,}667LR_d/A$, where R_d/A is the unit compressive stress on the pile. This stress will usually vary between 500 and 2,000 lb./per sq. in., the former representing easy driving and the latter very hard driving. If the mean cross section of the pile is 100 sq. in., these figures correspond to values of R_d of 50,000 and 200,000 lb., respectively. This expression for C_1 is strictly true only when the entire load is carried to the foot of the pile. Where the load is taken out entirely by friction uniformly distributed along the length of the pile, C_1 will be reduced one-half.

[1] Crandall, J. S., Piles and Pile Foundations, *J. Boston Soc. Civil Engrs.*, vol. 18, p. 176, 1931; "Concrete Piles," Portland Cement Association, 1936.

The value of C_2, which represents the rebound of the top of the pile due to soil elasticity, is difficult to evaluate, as it varies with the properties of the soil. For piles of usual size the following figures may be used: $0.00005R_d/A$ for firm gravel, $0.0001R_d/A$ for firm clay, and values as high as $0.00025R_d/A$ for soft ground.

The value of C_3, which represents the elastic compression of the pile cap, depends on the design and condition of the cap. If the packing is 6 in. thick and the bearing is on the sides of the fibers with an assumed modulus of elasticity of 100,000 lb. per sq. in., then $C_3 = 0.00006R_d/A$.

Hiley[1] gives the following average values for $C = C_1 + C_2 + C_3$:

TABLE 5-3c

Length of pile, feet	R_d/A = 500 lb. per sq. in. Easy driving			R_d/A = 1,000 lb. per sq. in. Medium driving			R_d/A = 1,500 lb. per sq. in. Hard driving			R_d/A = 2,000 lb. per sq. in. Very hard driving		
	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
10	0.19	0.16	0.25	0.28	0.21	0.41	0.37	0.27	0.57	0.41	0.27	0.67
20	0.23	0.19	0.28	0.36	0.27	0.47	0.49	0.36	0.65	0.57	0.39	0.79
30	0.27	0.22	0.31	0.44	0.33	0.53	0.61	0.45	0.74	0.73	0.51	0.91
40	0.31	0.25	0.34	0.52	0.39	0.59	0.73	0.54	0.83	0.89	0.63	1.03
50	0.35	0.28	0.37	0.60	0.45	0.65	0.85	0.63	0.92	1.05	0.75	1.15
60	0.42	0.31	0.40	0.68	0.51	0.71	0.97	0.72	1.01	1.21	0.87	1.27

(1) For timber piles.
(2) For reinforced-concrete piles with 1-in. material on head.
(3) For reinforced-concrete piles fitted with effective driving cap.

The Portland Cement Association[2] give the following simple method for finding $C_1 + C_2$ experimentally. A board is first clamped on the pile to which is attached a sheet of paper as shown in Fig. 5-3*a*. In front of this a straightedge is placed on a frame clear of the pile so that, when a pencil is drawn across the straightedge during driving, a record, as shown in Fig. 5-3*b*, is traced on the paper. From this diagram the permanent set and the temporary elastic compression below the point on the pile where the record was taken can be measured.

These equations may be used in their present form for drop-hammers and for single-acting steam-hammers. For double-acting

[1] Hiley, A., Pile-driving Calculations with Notes on Driving Forces and Ground Resistance, *Structural Engr., London*, vol. 8, p. 246, July, 1930.

[2] "Concrete Piles," Portland Cement Association, 1936.

steam-hammers the energy in inch-pounds developed by the steam must be added to W_hH. Tables showing energy ratings for double-acting hammers are given in the catalogues published by manu-

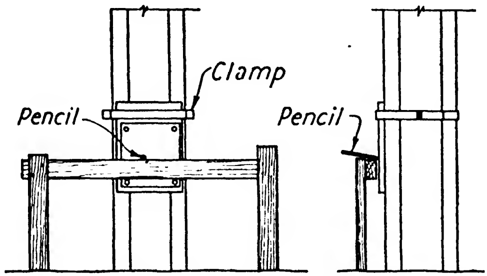


FIG. 5-3a.—Device Used for Finding Elastic Compression of Pile and Soil ($C_1 + C_2$).

facturers of steam-hammers. In using these tables care must be exercised to see that driving is done under conditions which will assure full energy development. The Boston Building Code specifies that "double-acting steam-hammers shall be equipped with an approved device for determining the actual energy in foot pounds per blow delivered."

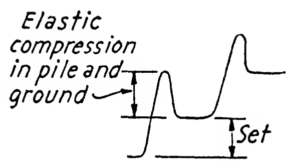


FIG. 5-3b. – Graph Showing Elastic Compression of Pile and Soil.

For further information on this subject the reader may consult the following valuable references:

Pile-Driving Calculations with Notes on Driving Forces and Ground Resistance, by A. Hiley; The Structural Engineer, London, vol. 8, page 246, July, 1930 and vol. 8, page 278, August, 1930.

Piles and Pile Foundations, by J. S. Crandall; Journal Boston Society of Civil Engineers, vol. 18, page 176, 1931.

"Concrete Piles," by Portland Cement Association, 1936.

5-4. Pile-driving Formulas and Applications. Many pile-driving formulas are in use in addition to the two developed in the preceding article. Most of these have a somewhat rational basis, with coefficients developed from tests. Such formulas are generally satisfactory when applied within the limits of conditions obtaining in the tests. Few of them may be considered to be of general application, as shown by the wide variation of results when they are applied to a given set of conditions. In 1902 Goodrich[1] developed a general formula and showed that 14 other pile-driving formulas could be derived from his general one by making certain

[1] *Trans. A.S.C.E.*, vol. 48, p. 180, August, 1902.

assumptions. Crandall[1] has also discussed the common basis of a number of the more commonly used formulas.

Four formulas are given below, the first two because of their sound rational basis and the second two because of their wide use in this country.

Let R_a = allowable load on pile in pounds
W_h = weight of moving part of hammer in pounds
W_p = weight of pile and cap in pounds
$r = W_h/W_p$
H_e = effective fall of hammer in feet ($= eH$)
W_hH_e = the actual delivered energy in foot-pounds per blow (including for double-acting steam hammer the work done by the steam pressure on downward stroke)
s = average penetration per blow in inches under last few blows
A = mean cross-sectional area of pile in square inches
L = length of pile in inches
E = modulus of elasticity of pile in pounds per square inch (1,500,000 for timber)

Taking Eq. 5-3*b*, which is the rational formula for ultimate resistance, and applying a factor of safety of three, we have, after multiplying the numerator by 12 (since H is expressed in feet with all other dimensions in inches)

Hiley:

$$R_a = \frac{4eW_hH}{s + \frac{C}{2}} \frac{r + n^2}{r + 1}, \tag{5-4a}$$

where e, C, and n are given in Art. 5-3.

The following pile-driving formula which appears in the Boston Building Code has the same form as the Hiley formula if n is assumed to be zero. For cast-iron on wood this coefficient of restitution is about 0.2 to 0.25, and, as it enters the rational formula as a square, the resulting error, in dropping it, is only 4 to 6 per cent.

Boston Code:

$$R_a = \frac{mW_hH_e}{s + K} \frac{r}{r + 1}, \tag{5-4b}$$

where m = 3 for drop-hammer
m = 3.6 for single-acting steam-hammer

[1] Crandall, J. S., Piles and Pile Foundations, *J. Boston Soc. Civil Engrs.*, vol. 18, p. 176, 1931.

$m = 4$ for double-acting steam-hammer

$K = \dfrac{1.5R_aL}{AE} + 0.05$ for a timber pile or wood driving cap

One of the oldest, simplest, and most widely used formulas is the *Engineering News* formula developed by A. M. Wellington for use with timber piles placed with a drop-hammer. It was later modified for use with a steam-hammer:

Engineering News:

$$R_a = \frac{2W_hH_e}{s + c}, \tag{5-4c}$$

where $c = 1$ for drop-hammer

$c = 0.1$ to 0.3 for steam-hammer

In the development of this formula, Wellington assumed that all of the hammer energy, $12W_hH_e$, when expressed in inch-pounds, went to overcome the driving resistance energy, Rs. Equating these two and using a factor of safety of 6, he got $R_a = 2W_hH_e/s$. When applying this formula to piles having a small penetration under the last few blows, it was seen that the results were absurdly high. To correct for this, $c = 1$ was added to the denominator. With the advent of the steam-hammer it was suggested that the coefficient be made 0.1 since the intervals between blows is very much shorter in the case of steam-hammers. Experience has indicated that 0.1 is too low a figure for small penetrations. In general, for penetration less than 0.15 in., $c = 0.3$ should be used. It will be observed that the *Engineering News* formula is of the Hiley form when $n = 1$.

In the early years, when all pile driving was done by drop-hammers on timber piles, the weight of the hammer usually equaled, or was somewhat greater than, the weight of the pile; hence r was more or less a constant. The percentage of the hammer energy delivered to the pile varies widely for varying values of r; consequently with the advent of heavy concrete and metal piles, it was found that the *Engineering News* formula gave bearing values far out of line with the results of static load tests. This formula was then modified to give

Modified *Engineering News*:

$$R_a = \frac{2W_hH_e}{s + \dfrac{c}{r}}. \tag{5-4d}$$

Let us apply these four formulas to a test made by the Missouri

Pacific Railroad[1] in 1932. In a static load test a timber pile carried a load of 88,000 lb. with a settlement of 0.66 in. In the driving test, W_h = 5,000 lb. (single-acting hammer), W_p = 2,520 lb., H = 3 ft., s = 0.75 in., L = 240 in., and A = 143 sq. in.

The constants in the Hiley formula are as follows: $e = 0.9$ (Table 5-3*a*); $n = 0.25$ (Table 5-3*b*); and $r = 5{,}000/2{,}520 = 1.98$. To find the value of C from Table 5-3*c*, we must first evaluate R_d/A. Assuming that R_a = 40,000 lb., then $R_d = 3 \times 40{,}000 = 120{,}000$ lb. and $R_d/A = 120{,}000/143 = 840$ lb. per sq. in. Interpolating in Table 5-3*c*, we have $C = 0.32$; hence

Hiley:

$$R_a = \frac{4 \times 0.9 \times 5{,}000 \times 3}{0.75 + \dfrac{0.32}{2}} \frac{1.98 + (0.25)^2}{1.98 + 1} = 40{,}600 \text{ lb.}$$

In the Boston Code formula, assuming R_a = 37,500 lb.,

$$K = \frac{1.5 \times 37{,}500 \times 240}{143 \times 1{,}500{,}000} + 0.05 = 0.113;$$

hence, Boston Code:

$$R_a = \frac{3.6 \times 5{,}000 \times 3 \times 0.9}{0.75 + 0.11} \frac{1.98}{1.98 + 1} = 37{,}500 \text{ lb.}$$

Engineering News:

$$R_a = \frac{2 \times 5{,}000 \times 3 \times 0.9}{0.75 + 0.1} = 31{,}800 \text{ lb.}$$

Modified *Engineering News:*

$$R_a = \frac{2 \times 5{,}000 \times 3 \times 0.9}{0.75 + \dfrac{0.1}{1.98}} = 33{,}800 \text{ lb.}$$

The reader is referred to Arts. 6-16 and 7-6 for the application of these formulas to concrete piles and to steel H piles, respectively.

5-5. Limitations in Use of Pile-driving Formulas. Care must be taken in interpreting the results of pile-driving tests, for as indicated elsewhere, the time element may have a great influence on the bearing capacity. The relation between immediate pile resistance and ultimate bearing capacity depends primarily on two factors, (*a*) type of soil and (*b*) amount of water carried by the soil. Soils may be divided into two general classes, (*a*) the noncompressi-

[1] See *Am. Railway Eng. Assoc. Bull.* 349, p. 71.

ble sands and gravels and (*b*) the highly compressible and plastic clays, loams, and silts. However, in many cases these two general classes are more or less intermixed to form soils ranging all the way from those of little compressibility to those highly compressible. Then again in driving a pile we may pass through a number of different strata, which often makes it difficult to interpret test results.

Pile-driving formulas are applicable to sand and gravel (materials possessing high internal friction without cohesion), but they are not applicable to clays, loams, and silts (materials which have high cohesion). The bearing capacity of a pile depends on two different factors, namely, the frictional resistance acting along the sides of the pile and the point resistance, or the resistance of the soil against being compressed and displaced by the pile. In sand these resistances are reasonably constant, as they are not dependent on the time element; hence the resistances acting while the pile is being driven closely resemble those acting on the pile under static load. However, in fine sands saturated with water, the sides of the pile may be lubricated by a quicksand action, to give a decreased resistance. If the pile is allowed to rest for a short time and then driving is started again, the true static resistance may be obtained, as the sand quickly settles around the sides of the pile. The point resistance is very high in noncompressible soils because the soil which must be displaced to make room for the pile cannot be compressed into a smaller volume and, lacking plastic characteristics, cannot easily be laterally displaced. For these reasons the water-jet process by which the material is floated to the surface is a necessary adjunct to placing piles in noncompressible soils.

Experience has shown that, in driving piles through compressible soils, the ease or difficulty of driving bears little relationship to its ultimate load-carrying capacity. Unlike a sand, in which the water is squeezed out rapidly on application of load, a clay may be nearly impermeable and so the changes in volume under load application may proceed at a slow rate. In driving a pile, the impact of the blows and springing action of the pile tends to attract the water from the soil and thoroughly lubricate the sides of the pile, thus greatly reducing side friction. On the other hand, point resistance may be very large at the time of driving, owing to the time element required to squeeze out the water and compress the soil. So we may have a peculiar situation in which side friction may increase after the pile is driven and point resistance decrease. The increase in side friction is due to a gradual reabsorption of the free water into

the surrounding soil. This usually proceeds at a much faster rate than does the exit of water in the soil developing point resistance.

A practical test for determining whether the soil is of the first or second type is to compare the penetration under blows immediately before and after a rest period of at least 24 hr. If they are practically the same, the soil is of the first type and pile formulas may be used with assurance.

Formulas for the bearing power of piles are not designed for the case where the pile is driven through soft, yielding material to a hard stratum of compacted sand, gravel, hardpan, or rock, for then it acts like a column and must be designed as such.

The value of the penetration to use in the formula is generally taken as the mean of the last 5 or 10 blows. Where the drop-hammer is employed, no value less than $\frac{1}{4}$ in. should be considered in any case for timber piles—and usually not less than $\frac{1}{2}$ in. Even the average penetration under the last five blows is not a fair measure of the bearing power of the pile at that time, unless the penetration has been either uniform, or decreasing at an approximately uniform rate.

Whenever a follower has to be used on top of a pile as it is driven home and it is desired to compute its bearing power, it is necessary to apply a correction to the observed average penetration. For this purpose some tests should be made under fairly comparable conditions when each pile may be driven alternately with and without a follower. In those formulas having a term representing the weight of the pile, the weight of the follower should be added. Also for the Hiley and the Boston Code formulas, to the term representing the elastic deformation of the pile, the elastic deformation of the follower should be added.

5-6. Effect of Rest on Bearing Power. In some kinds of materials, like sand or gravel, if a pile be partly driven one day and driving is resumed the next day, the resistance as measured by the average penetration per blow at the beginning of the second day's driving is generally found to be practically the same as that at the close of the first day. However, this is not always true, even in sand. Occasionally the resistance is found to decrease. Terzaghi[1] cites a case as follows:

On the south side of the Island of Galveston, to a depth of about 50 ft., the underground consists of very fine sand mixed with powdered shells. Because of the very dense structure of this sand, it is considered impossible

[1] *Trans. A.S.C.E.*, vol. 93, p. 387, 1929.

to drive piles to a depth of more than 15 or 20 ft. without smashing the pile head. According to Colonel Schley, Government Engineer of Galveston, some piles driven with a steam-hammer to refusal settled afterwards several inches under a static load of less than 15 tons.

Such an occurrence is clearly a matter of stress relief or decompression. Terzaghi states that there seem to be no records of conspicuous decompression phenomena except for cases in which the pile driving was done with a steam-hammer that could apply pressure very rapidly. The intensity of the decompression phenomena may be considerably increased by the presence of a layer or packet of very loose sand a short distance beneath the point of the pile.

A more usual phenomenon in driving through sand is to find an increase in resistance after a period of rest, due, as stated in the previous article, to the sand settling more firmly around the sides of the pile.

The effect of rest on piles driven into compressible and plastic soils is very difficult to predict, although it is often very striking. Increases of resistance have been observed amounting to as much as tenfold in 24 hr. On the other hand substantial decreases have also been observed.

An old contractor reports an illustration of increased resistance. An attempt was made to reach hard bottom through a very deep marsh. After driving a 35-ft. pile, another one of equal length was spliced to it and also driven without finding a hard stratum, sinking 4 or 5 in. per blow. Then a second pile was started. After driving this pile to a penetration of 25 ft., it was time to quit work; hence the pile was left in the leads in that position. The next morning five or six blows of the hammer failed to produce any appreciable movement. Accordingly the engineer decided to drive piles about 30 or 40 ft. long and to depend on the friction in the soft meadow muck to support them. The trestle bridge, thus supported, carried its traffic safely for years, the locomotives and cars becoming much heavier than it was originally expected would ever be used on it.

At Fort Point Channel, Boston, borings 65 ft. below low water showed only soft blue clay changing in a few instances to soft yellow clay. Tests were made to determine the increased frictional resistance after a period of rest. A spruce pile 35 ft. long, 17 in. at the butt, and 7 in. at the tip was driven 20 ft. into the clay by a 2,360-lb. hammer falling 8 ft. The average penetration for the last 5 blows was 5.5 in. After a 4-day rest the pile was struck 20 blows, giving an average penetration of 0.9 in. for the first 5 blows and 1.5 in. for

the 20 blows. With another pile in slightly softer material the average penetration for five blows before and after 4-day rest decreased from 7.6 to 1.6 in.

In the upper San Francisco Bay there is a deposit of very soft silt due to the finest and lightest tailings from hydraulic mining in the Sacramento and San Joaquin rivers. A pile sinks 20 to 27 ft. of its own weight, but with a total penetration of 40 to 55 ft. will later support 40,000 lb. If a pile is allowed to rest only 15 min., a heavy blow from a drop-hammer will not move it perceptibly, but a few blows in succession will start it at the old rate. While attempting to splice a pile on one occasion, the mud settled against it so that it could not be driven farther and a scow with a displacement of 30 tons could not lift it. The friction developed was 200 lb. per sq. ft.

Experiences showing a decreased resistance are not so numerous or so marked. In the construction of the extension of Concrete Pier No. 314, Navy Yard, Charleston, S. C., a number of 18-in., square, wooden test piles were driven through 10 ft. of silt and into stiff blue mud. At a penetration of from 43 to 57 ft. the test piles developed a resistance to driving of 40 tons and soon thereafter reached the refusal point. After a rest of from 3 hr. to 2 days a marked decrease in resistance appeared, this varying for the several piles as follows: from 43 to 21 tons, 45 to 32 tons, 47 to 39 tons, and 39 to 25 tons.

5-7. Spacing of Piles. When the bearing power of the earth chiefly supports the pile at its foot, it should be designed as a column. and the piles may be spaced as closely as it is practicable to drive them. In good practice, piles are never spaced closer than 2½ ft. between centers and preferably not closer than 3 ft.

When a pile is held in position entirely by frictional resistance, the load is transferred to the adjacent ground and transmitted down to different levels in widening areas until a level is reached where the earth can readily support the unit bearing value. The mass of earth thus transmitting the load may be said to form approximately a conoid of pressure, the slope of which depends on the nature of the soil with respect to both the kind of material and its degree of compaction. When the frictional resistance is small, the total penetration must be larger, or the pile will settle under the load, sometimes slipping through the surrounding material and at other times carrying a mass of earth with it.

There is little information available on friction values, for in all static and driving tests the total resistance includes both friction and point bearing, and it is very difficult to separate the two effects.

The best method is to test for pulling resistance. Tests[1] indicate that the frictional resistance may vary from a little over 100 lb. per sq. ft. of pile surface in the case of soft mud to 2,000 lb. per sq. ft. for compact sand. Tschebotareff gives the following average values, expressed in pounds per square foot: marsh, 140; silt, 360; sandy clay, 600; loose sand, 700; stiff clay, 1,500; and compact sand, 2,200.

Sometimes piles are driven with close spacing in order to compress the soil and thus increase its load-carrying capacity. The desired compression may not always be obtained, for the displaced soil may come up along the sides of the pile, or general heaving of the soil may result, in which case the level of the ground is raised. The possible consolidation of the earth depends largely on the amount of water and air contained therein, that is, on its percentage of voids. Loose dry bank sand contains about 35 per cent voids. If water is added, the voids will increase to a maximum of approximately 45 per cent, at which point the percentage of water by weight based on the dry sand will be about 7 per cent. As more water is added, the voids decrease to a minimum of about 33 per cent for a percentage of water of 17 or more. For a thoroughly tamped sand these percentages are, respectively, about 27, 31, and 26½.

Generally speaking, sands cannot be consolidated unless they contain percentages of water within the bulking range. Because of their high coefficient of internal friction sands cannot easily be displaced laterally. On the other hand, compressible and plastic soils can be consolidated to a considerable degree but only at a very slow rate, for owing to their impermeability the resistance to the escape of water is high. Owing to their plastic nature, clays can be readily displaced in driving piles to cause a bulging of the surface.

It has been noted that when piles are driven into the soft clays of Boston, the surface of the ground between the piles will rise to a height of several inches, indicating that the volume of voids remains practically unchanged. In fact the pile driving seems to make the ground softer. On the other hand, it has been observed that when piles are driven into exceedingly fine-grained quicksand, the surface between the piles actually subsides, sometimes almost a foot, indicating a very substantial consolidation of the soil.

Strohchneider's modification of Boussinesq's analysis indicates that, for any pile in which the load is carried by friction alone, the load is distributed out at the foot of the pile over a horizontal

[1] *Trans. A.S.C.E.*, vol. 103, p. 1200, 1938; "Concrete Piles," Portland Cement Association, 1936.

circular area, the diameter of which is approximately $1.2L$, where L is the length of pile embedment. The intensity of pressure varies from a maximum of $3.5R/L^2$ directly under the pile to zero at the circumference of the circle, R being the pile load. With this wide distribution of stress it is evident that with close spacing of piles there is a marked overlap of zones of stress, the amount of overlap increasing with a decrease in pile spacing, as well as with an increase in pile length. The effect of loading on a group of 25 such friction piles[1] spaced a distance apart equal to $0.2L$ is shown in Fig. 5-7*a*. The dotted lines are the vertical stress curves for the individual piles. The full line is the combined curve of vertical stress under the

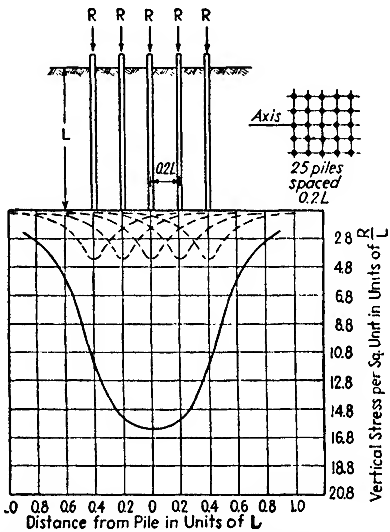


FIG. 5-7*a*.—Friction Piles in Group Showing Distribution of Loads to Soil.

center row of the group, indicating a maximum stress of $15.4R/L^2$. Compared with a value of $3.5R/L^2$ for a single pile, this shows the very marked overlapping effect of the group.

If a pile foundation is placed in soft material of great depth, it is important that in addition to static tests on a single pile, such tests should also be made on a group of four or more. In this way valuable information will be obtained on the actual overlapping effect. To assume that, if a single pile can support a load of 15 tons in this type of soil with a given settlement, 20 piles will support a load of 300 tons with the same settlement may be a serious error.

[1] Crandall, J. S., Piles and Pile Foundations, *J. Boston Soc. Civil Engrs.*, vol. 18, p. 176, 1931.

5-8. Degree of Security. It is of great importance in designing a pile foundation that the proper degree of security or factor of safety be chosen. In the Hiley formula (Eq. 5-4*a*) for determining bearing power from the dynamic resistance in driving, a factor of safety of three is presumed, while in static test specifications the permissible load is usually limited to about one-half the load causing a small settlement, usually about $\frac{1}{2}$ in. In selecting so-called safe loads, the problem is to limit the load to a value for which the settlement will be within permissible limits. A reasonable degree of security must be used to take care of the many uncertainties involved. Granted that the tests give a true value of the carrying capacity at the time of testing, some of the uncertainties for which allowance must be made are as follows: (*a*) effect of time on pile resistance, (*b*) effect of group action as compared with a single pile, and (*c*) possible increase of live loads.

As explained in preceding articles the bearing power of a pile is quite likely to change over a period of time. Generally it increases, but in certain types of soils it may decrease. A change in the moisture conditions due to hydraulic constructions may transform a stratum of clay into a slowly yielding mass which permits piles to sink into it that before appeared to have a solid support; or deformation in the ground of a contiguous site may reduce the bearing power at a given point. The effect of time in reducing the carrying capacity of piles driven into clay is especially to be feared where the point bearing is large owing to the gradual consolidation of the material as water is slowly forced out, this action often extending over a period of years.

The combined resistance of a group of piles is usually less than the sum of the individual carrying capacities, particularly in deep plastic soils. No exact figure can be given as to the amount, for it is not possible to test a large group of piles as a unit.

In some cases most of the load to be supported by piles is dead load, while in other cases it is live load. The live load may increase during the life of the foundation, for example, that of locomotives and trains passing over a railroad structure. In addition to the static weight of the live load, some provision must often be made for the dynamic effect due to a moving load or to the vibration of machinery. At a factory on Fort Point Channel, Boston, the movement of piles may be observed when the machinery is in motion. The allowance thus made is usually less for substructures than for superstructures. If a building is adjacent to a railroad, some account must be taken of the fact in designing the pile foundation.

In some structures, like that of a wharf, the failure of piles may cause serious loss of property or even loss of life; hence a larger margin of security is needed.

A building may be subsequently used for a different purpose than for which it was designed. The building itself may be readily strengthened, but it is usually impracticable to increase the strength of the foundation because of excessive cost. Such contingencies are provided for only in special cases, but it is well to keep them in mind.

An ample degree of security should always be provided for and the following contingencies considered: (*a*) foundation problems are less susceptible of a rational analysis than those of any other part of a structure, (*b*) bearing soils are variable and uncertain in action, (*c*) a foundation failure usually affects the whole structure, and (*d*) it is usually very difficult and expensive to make foundation renewals.

5-9. Lateral Resistance of Piles. Where lateral loads of considerable magnitude are present, batter piles should be used. However, vertical piles driven well into compact soil offer considerable resistance to lateral forces. A theoretical analysis of lateral load resistance is rather complicated and, lacking coefficients based on actual tests, is of little value. For such analyses the reader is referred to the discussions in the paper noted in the next paragraph.

One of the few series of tests on lateral resistance of timber and concrete piles was made at Lock and Dam No. 26, Alton, Ill., the results of which were given in a paper by Lawrence B. Feagin.[1] The material penetrated was medium sand. The program included tests on a single timber pile and on a single concrete pile, as well as on piles in groups numbering 4, 12, and 20 piles. The single piles were tested without top restraint, while the pile groups were tested with the pile tops embedded 2 ft. in 6-ft. monoliths of concrete. In most cases the load was applied in cycles, the load being completely removed after each application.

In the single timber-pile test, the pile was 32 ft. long, with a tip diameter of 10 in. and a butt diameter at ground level of 14 in. The penetration of the pile was 30 ft. Driving was done with a single-acting steam-hammer having a ram weight of 5,000 lb. and a stroke of 35 in. The penetration for the last 100 blows was 1 ft., which gives a safe bearing value by the Boston Code formula (Art. 5-4), on the basis of $W_p = 1{,}600$ lb. and $A = 122$ sq. in., of 83,000 lb.

A 350-ton capacity hydraulic jack was used to develop the lateral force which was applied at a point 14 in. above ground surface, the

[1] *Trans. A.S.C.E.*, vol. 102, p. 236, 1937.

jack reacting against a second pile. The lateral deflection at the top of the pile varied uniformly as the load was increased up to a load of 6 tons, at which point the deflection was $\frac{5}{8}$ in. At this point there was a flattening of the load-deflection curve (loads plotted as ordinates and deflections as abscissas) and at a load of 16 tons the deflection was $2\frac{1}{4}$ in., with eight cycles of load application.

A similar test was made on a concrete pile with a tip diameter of $10\frac{3}{4}$ in. and a butt diameter at ground-line of 18 in., the length of the pile and the depth of penetration being the same as for the timber pile. The average penetration per blow under the last few blows was 0.076 in.; hence the safe bearing capacity by the formula noted in the preceding paragraph, based on $W_p = 6{,}000$ lb., $A = 184$ sq. in. and $E = 2{,}000{,}000$ lb. per sq. in., is 84,000 pounds. The load-deflection diagram had a constant slope up to a load of 8 tons, at which point the deflection was $\frac{3}{8}$ in. At this point the curve flattened somewhat, and for a load of 18 tons the initial deflection was about $\frac{7}{8}$ in., which increased somewhat without any increase of load.

In testing one monolith having four timber piles and one having four concrete piles, the stiffening effect of fixing the pile top in the concrete was plainly evident. For example, with a total load of 40 tons, or 10 tons per pile, the lateral deflection for the timber piles was $\frac{3}{8}$ in., as compared with a 6-ton load causing a deflection of $\frac{5}{8}$ in. for the single pile test with head of pile free. Likewise, with a total load of 54 tons, or $13\frac{1}{2}$ tons per pile, the lateral deflection for the monolith of concrete piles was $\frac{3}{8}$ in., as compared with an 8-ton load causing the same deflection for a single concrete pile with head of pile free.

As a result of these tests it was concluded that for the piling arrangements and type of soil in which these tests were made, the following lateral loads are permissible: Timber piles for which a deflection of not more than $\frac{1}{4}$ in. is permissible, 4 tons per pile if the piles are subject to frequent repetition of load, or $4\frac{1}{2}$ tons per pile if subject only to sustained load. Where a deflection of $\frac{1}{2}$ in. is permitted, these values may be increased to $6\frac{1}{2}$ and 7 tons, respectively. For concrete piles these values may be increased from $1\frac{1}{2}$ to 2 tons per pile.

5-10. Uplift Resistance. It is sometimes necessary to design timber piles for uplift forces, for example, in California, where the laws require buildings to be designed to resist earthquake shocks. These shocks may subject all structural elements to abnormal strains. Uplift forces also often result from hydrostatic pressures

and from wave action. Sometimes bridges must be designed to withstand the action of hurricanes, where the overturning moment of the wind forces creates uplift over a part of the foundation piers.

Failure of a pile foundation from uplift forces may result either from movement of the piles in the ground or from the concrete footing pulling away from the pile tops. Generally uplift forces are not great enough to cause a tension failure in the piles. Since pull-out resistance depends entirely on side friction, whereas bearing capacity depends on both side friction and point bearing, generally the former will be less than the latter. As stated in Art. 5-7, tests indicate that frictional resistance may vary between the wide limits of 100 and 2,000 lb. per sq. ft.

In designing a building[1] for Sears, Roebuck and Co. in Los Angeles, the piles were relied on for a reaction against uplift of about 7 tons each, the downward permissible load being 16 tons each. The piles were driven to a penetration of about 20 ft. through clay, sandy clay, and sand. Two piles were selected for a settlement test under gravity loads. The average penetration per blow for the last few blows of a 3,000-lb. hammer falling 10 ft. were 0.3 in. for the first pile and 0.4 in. for the second pile, indicating by the *Engineering News* formula safe bearing capacities of 23 and $21\frac{1}{2}$ tons, respectively. The two test piles were loaded to 25 tons each and no progressive settlement occurred for the 30 hr. during which this load was applied. The movement in each case approximated the elastic deformation to be expected and was less than 0.08 in.

Two piles were also subjected to pull tests. The maximum load applied to one pile was 33 tons, the concrete cap through which the load was applied failing at this point. The second pile was loaded to 75 tons, the deflection being 0.2 in., which decreased to 0.04 in. on release of load, this indicating that there was virtually no movement of the pile in the earth.

Results of a set of tests made on the bonding of pile heads encased in concrete appears in *Public Roads*, vol. 9, p. 169, November, 1928. The piles were of southern pine $7\frac{3}{4}$ to 11 in. in diameter, the embedment in the concrete being 20 in. All but four of the pile specimens were lathe turned. One series of tests was made with concrete cast in the dry, while a second series was made with concrete placed through water. The results were shown in the table on page 159.

The authors expressed the opinion that the results of the tests where the concrete was placed under water were lower than might be

[1] *Wood Preserving News*, vol. 18, p. 28, March, 1940.

Pile	Concrete placed	Age of concrete	Load at initial slip, pounds		Load at initial slip, pounds per square inch		
			Maximum	Minimum	Maximum	Minimum	Average
Turned	Dry	7 days	27,300	18,730	52	33	42.5
Turned	Dry	21 days	38,095	25,715	68	44	59.5
Unturned	Dry	7 days	61,905	35,873	96	62	79
Unturned	Dry	21 days	43,808	27,936	63	49	56
Turned	Under water	21 days	11,114	15,140	21	28	25

expected in practice. In the first place no additional cement was used, contrary to usual practice. In the second place the depth of water was small; hence the concrete was not subjected to the compacting pressure usually present where a mat of concrete is placed around pile heads.

This series of tests also included experiments in which the object was to increase the pull-out resistance by (*a*) expanding the diameter of the pile heads by wedges and (*b*) driving spikes into the sides of the piles with a part of their length embedded in the concrete. Neither method resulted in any substantial increase in pulling resistance.

CHAPTER VI

CONCRETE PILES

6-1. Introduction and Classification. Being impressed by the very short life of timber piles when their upper portions are alternately wet and dry (as in pile trestle bridges), the increasing cost of timber, and the decreasing cost of cement, A. A. Raymond was led to consider the design and construction of concrete piles. He first used such piles in 1901 in a building foundation in Chicago.

In 1897, Hennebique introduced the reinforced-concrete pile in Europe, and in 1904 it was first used in America. Within the first decade of this century, a number of other forms were developed, differing in methods of construction. Some of them were patented by the inventors, whereas others were designed by engineers, without the use of patented material, form, or arrangement.

Concrete piles may be divided into two general classes: The first class comprises those which are molded to a regular form and, after curing, are handled and driven like timber piles; while the second class includes those formed in place either with or without the use of casings which remain until destroyed by corrosion. The former may be called precast piles and the latter, cast-in-place

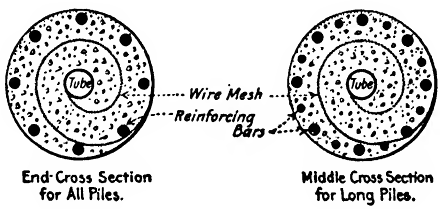


FIG. 6-1a.—Sections of Chenoweth Pile.

piles. Precast piles were first developed in Europe, and practice in that country has practically confined the use of concrete piles to that class. The cast-in-place piles were invented in America and on that account came into considerable use before the advantageous features of precast piles came to be generally recognized.

Precast piles, rather than cast-in-place piles, are usually used in docks, piers, wharves, concrete-trestle bents, and similar structures where a portion of the pile remains above firm surrounding

soil and where considerable bending moment is likely to develop in the pile. Precast piles are always reinforced with steel bars or rods in combination with lateral reinforcement in the form of wire hoops or spiral wrapping. They are square, octagonal, or circular in cross section, the corners of square piles being chamfered, however.

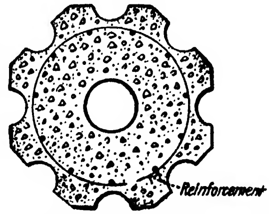


Fig. 6-1b.—Section of Corrugated Pile.

In the early days a number of precast types were patented, but most patents have now expired. For example, the Chenoweth pile (Fig. 6-1*a*) is formed by rolling concrete in a machine specially designed for the purpose, the reinforcement being arranged to show a spiral form in cross section. The corrugated pile (Fig. 6-1*b*) is octagonal in section but with a semicircular

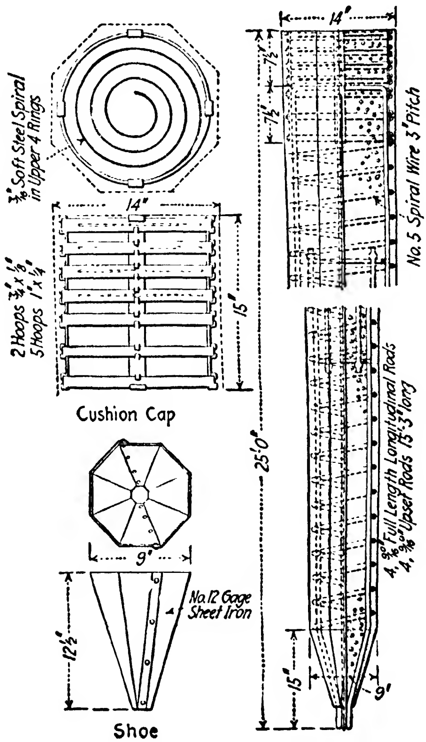


Fig. 6-1c.—Reinforcement of Cummings Concrete Pile.

corrugation on each face to increase the surface for frictional resistance and with a hole along its axis. The Cummings pile (Fig. 6-1*c*) is distinguished mainly by the character and arrange-

ment of the reinforcement, which is electrically welded and handled as a unit. In some cases annular grooves are molded on the surface.

Cast-in-place piles are well suited to soils having compressibility together with sufficient stability to prevent damage to the soft concrete. This type of pile is generally not reinforced, although this feature may be added. Among the types of cast-in-place piles in extensive use are the Raymond, the Simplex, the MacArthur, the Franki, and the Monotube.

In the following articles of this chapter will be found descriptions of the several types of concrete piles now in common use as well as information on design and construction. For complete specifications the reader is referred to the Manual of the American Railway Engineering Association, to the Standard Specifications for Highway Bridges of the American Association of State Highway Officials, and to city building codes such as those of Boston or New York.

6-2. Relative Advantages. Untreated timber piles in ordinary foundations must be cut off below the permanent ground-water level, which often involves the cost of extra excavation. Where the water level is lowered by changes in the drainage system due to the construction of subways, the lowering of sewers, or for other causes, the piles become liable to rot and may involve expensive changes in the foundation. In Art. 4-9 reference is made to the failure of a timber-piling foundation in San Francisco due in part to the lowering of the water level and in part to overdriving of piles. When the building was constructed, the water table was only a few feet below ground surface. Subsequently large buildings were constructed on all sides, in many instances involving open foundation excavation unwatered to a considerable depth. In addition, complete surface coverage of paved streets and roofs contributed to a recession of the water level, until finally a drop of more than 9 ft. took place.

The durability of concrete piles is independent of the ground-water level. Concrete piles have a material advantage on account of their greater bearing capacity, due to their larger size, thus permitting a material reduction in the number required to support a given structure. Roughly, the loading of timber piles is restricted by engineers within a range of 10 to 25 tons, while concrete piles may be loaded from 20 to 75 tons.

On the other hand, a concrete pile costs considerably more than a timber pile, approximately $2 to $4 per foot in place as compared with 35 to 80 cts. per foot for untreated timber piles and 75 cts. to $1 per foot for 12-lb. treatment creosoted timber piles. Since the life of concrete piles is not dependent on the ground-water level,

their use not only avoids extra excavation but as a direct consequence a saving in the footings or masonry walls. Concrete piles may also be readily bonded into the grillage or capping of concrete to form a monolith, provided reinforcement is used in the piles. Less excavation and smaller footings imply a reduction in the time of construction.

When precast piles are employed, they require more time and care in handling than do timber piles, on account of their greater weight and their relatively lower flexural strength. In general, concrete piles cannot be driven so rapidly as timber piles, but the number required may be sufficiently smaller to effect a saving in time, as well as in cost. As cement, sand, and stone are generally available, there is less probability of delay, occasioned by waiting for the arrival of piles at the site. Sometimes the value of the time saved pays for a considerable part of the piles. In some track-elevation work in cities the use of concrete piles for the foundations of retaining walls has made a large saving by reducing the required width of new right of way at the excessive rates that had to be paid.

In emergencies concrete piles have sometimes shown unusual flexural strength. In one case a single pile acting as a cantilever, 32 ft. long, successfully withstood the test when a 9-in. hawser was attached to its upper end to pull a steamer of 4,800-tons displacement to the unfinished pier against a rapidly running tide. At another time a steamer ran into a pier by accident and broke off a number of pine piles but none of the concrete piles. It may be added that concrete piles may be placed in some filled material through which it is impossible to drive timber piles without injury. Strict economy requires that adequate exploration of the soil be made to determine the proper lengths of piles. Failure to do so leads to waste of timber by excessive cutoffs, but with concrete piles the waste of time may be even more serious than that of material and labor.

Inspectors realize how difficult it often is to find a fair percentage of timber piles which fill the demands of the specifications in all particulars—such as diameter of butt, diameter of tip, straightness, and other qualities—especially when the required length exceeds 50 ft. As the forest resources are being reduced, it becomes increasingly difficult to get the larger sizes of timber piles, and at the same time the quality of the wood becomes poorer. On the other hand, with reasonable care, every concrete pile can be made to comply fully with the specifications. Even though the safe allowable unit

compression for concrete is less than for wood on the ends of the fibers, this may not be of much importance, as the load capacity of a pile frequently depends on the supporting capacity of the earth rather than on the strength of the pile.

In salt water infested by marine borers, timber piles require expensive protection either by chemical treatment (Art. 4-18) or by mechanical means (Art. 4-19). Concrete piles are free from the ravages of these borers, but, on the other hand, the problem of disintegration of the concrete by chemical action of the sea water and by mechanical wear is encountered.

6-3. Precast Piles. A precast pile is a reinforced-concrete pile which is molded to a regular form and, after curing and seasoning, is handled and driven like a timber pile. In order to indicate the principal variations in form and reinforcement which have been

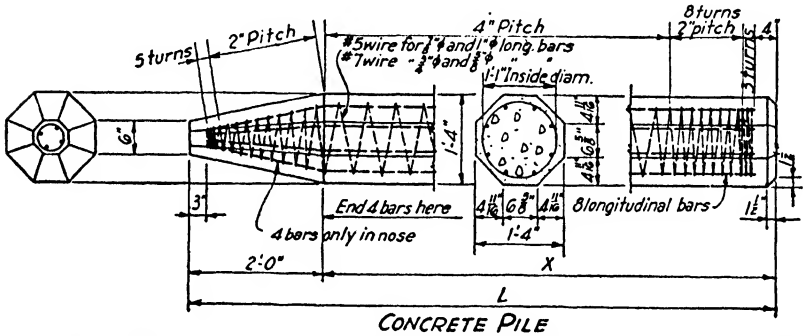


FIG. 6-3a.—Standard Reinforced-concrete Pile of American Railway Engineering Association.

developed by different designers of precast concrete piles, brief descriptions are given of some typical designs.

Figure 6-3a illustrates the standard 16-in. precast concrete pile designed for a load of 30 tons for concrete-trestle construction of the American Railway Engineering Association. This design is of a four-pile bent for Cooper's E60 loading, the bents being spaced 16 ft. centers. The longitudinal reinforcement consists of eight bars varying from $\frac{5}{8}$-in. round bars to 1-in. square bars, depending on the length of the pile and the bending moment. For further details of this design, as well as for the details of 24-in. piles designed for loads of 75 tons, the reader is referred to the Manual (1937) of this association.

The reinforced-concrete piles used by the Pennsylvania Lines for their extensive docks at Cleveland, Ohio, were octagonal in shape, were without taper, were pointed at the foot, and had a

cast-iron shoe which was made an integral part of the pile. As indicated in Fig. 6-3*b*, the reinforcement consisted of eight longitudinal rods securely bound together at regular intervals throughout the body of the pile by the rods. They were also spirally wrapped for short distances at both head and foot. The dimensions in Fig. 6-3*b* refer to piles 30 to 40 ft. long, the longitudinal rods being 1 in. in diameter, while the ties and wrapping were $\frac{3}{8}$ in. in diameter. These piles were cast in a vertical position.

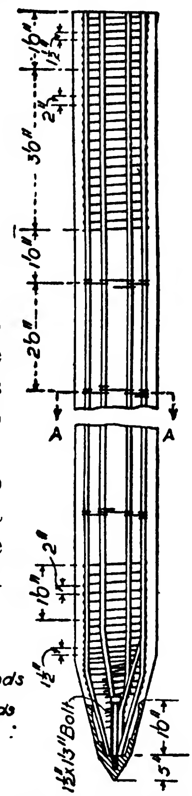


Figure 6-3*c* shows the design of an 85-ft. pile cast in a horizontal position and used in dock construction in Havana. The section is 20 in. square, the reinforcement being proportioned for the bending moment when the pile is supported at two points located to make the maximum and negative moments equal, namely, the one-fifth points. Transverse gas pipes were inserted at these points to provide a convenient method of handling the piles. The piles were hooped with $\frac{1}{4}$-in. square rods and had a special basket reinforcement at the top to take the hammer impact.

In building a bridge in New Zealand where no highly resistant stratum could be reached at a reasonable depth and so dependence had to be placed on friction alone, the 30-ft. piles were tapered to transfer the load more certainly and were made square in section to give increased surface area. The cross section varied from 16 to 12 in. square. The lower end was pointed.

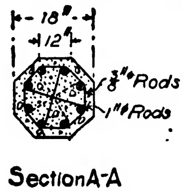


FIG. 6-3*b*.—Reinforced-concrete Pile Used in Dock Construction in Cleveland.

Concrete piles used for a concrete-trestle bridge over a tidal stream at Yarmouth, Mass., were formed with a 2-in. thickness of Gunite—a patented form of Shotcrete (Art. 6-14)—for a length of 9 ft., centered at mid-tide to resist the disintegrating action of the sea water. The piles, which varied in length from 37 to 48 ft., were 16 in. square in cross section, with 2-in. chamfered edges. They were pointed to an 8-in. square section at the tip, starting back 4 ft. The reinforcement consisted of eight longitudinal 1-in. round bars, tied together with $\frac{3}{8}$-in. round bars in the form of hoops. In casting the piles, the 9-ft. section containing the Gunite was

recessed 2 in. so that, when the Gunite was applied, its surfaces came flush with those of the rest of the pile.

Figure 6-3*d* shows a 24-in. square pile being driven for the Woodhaven Boulevard extension in New York. The maximum length of pile on this job was 105 ft. and the maximum weight 30 tons. The reinforcement consisted of $1\frac{1}{8}$-in. square bars and $\frac{1}{2}$-in. round bars, the volume of steel being 1.87 per cent that of the concrete. Probably the record length of concrete piles is 115 ft., 24-in. square piles of this length having been used in bridging the James River near Newport News.

The Jones-Bignall pile, a patented type, is designed to be placed without driving, sinking being effected entirely by the water-jet.

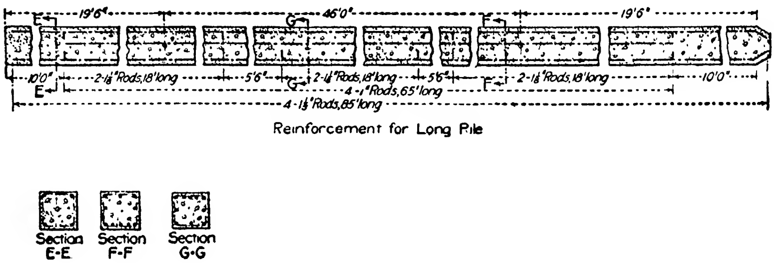


Fig. 6-3*c*.—Design of 85-foot Pile.

This pile has a 4-in. pipe through its entire length, this diameter being reduced by means of a special nozzle to $1\frac{1}{8}$ in. at the bottom. This 4-in. pipe is tapped at intervals throughout its length with pipes of from $\frac{3}{4}$- to $\frac{3}{8}$-in. diameter, the latter being fitted with elbows turned up on the sides of the piling. In addition, a 2-in. pipe extends through the 4-in. pipe and fits into the $1\frac{1}{8}$-in. nozzle at the bottom. The 2-in. pipe carries a water pressure of 200 to 300 lb. per sq. in., while the larger pipe carries a pressure of about two-thirds this. The jet at the bottom of the pile loosens and displaces the material under the pile point, and the jets on the sides keep the soil loose and overcome side friction.

The piles used for the elevated-railroad approach to the McKinley bridge in St. Louis were made by a patented centrifugal process. By this process the concrete in special cast-steel molds was rotated in a horizontal position at 500 r.p.m. for 15 min. to form a dense hard concrete shell, leaving a smooth central hole filled with excess water from the mix. After spinning, plugs were removed at both ends of the mold and the excess water was drained off. The mold was then placed in a hot-water bath at a temperature of 175°F. for a curing period of 3 hr. The pile was then sufficiently cured to

permit removing the mold and handling. It was then further cured in a saturated atmosphere for 28 days before driving. The average

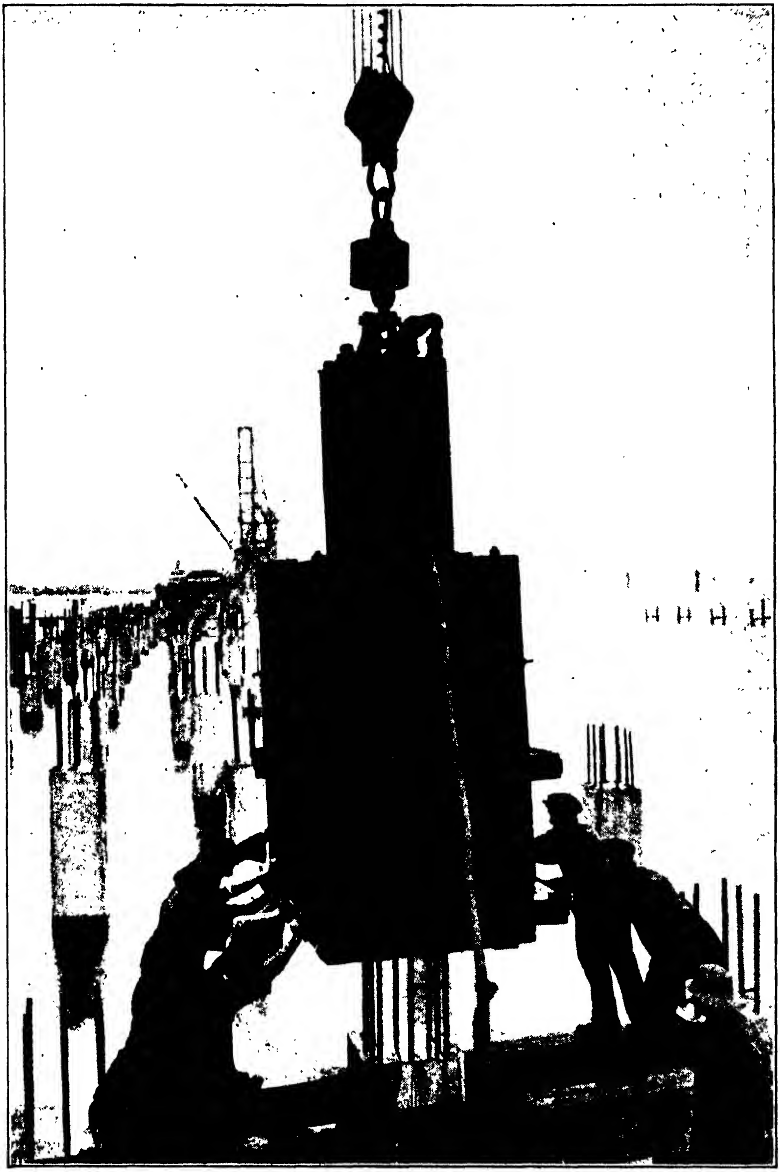


FIG. 6-3d.—Driving 30-ton Piles.

compressive strength of the piles was 5,000 lb. per sq. in. of cross section.

The piles varied from 20 to 40 ft. in length and were made with a taper of $\frac{1}{4}$ in. per ft. The tip and butt diameters, the thickness of the concrete shell, and the amount of reinforcement varied with the length of the pile. For example, one set of dimensions for a 30-ft. pile are as follows: tip diameter, $8\frac{1}{4}$ in.; butt diameter, $15\frac{3}{4}$ in.; wall thickness at tip, $2\frac{5}{8}$ in.; wall thickness at butt, 3 in.; reinforcement, six $\frac{5}{8}$-in. rods surrounded with spiral wrapping.

6-4. Form and Construction. The prevailing form of cross section for precast piles is octagonal. Practically all those with square sections approximate the octagonal form in that their edges are beveled to a width of 2 or 3 in. The circular cross section is used but seldom, in which case the pile must be cast in a vertical position. The diameter varies from 10 to 25 in., popular sizes being 16 and 18 in. The Manual of the American Railway Engineering Association specifies that "piles of constant cross section shall have a least diameter or lateral dimension of 14 in. for piles up to 35 ft. in length and not less than 16 in. for lengths over 35 ft. and not over 50 ft." The length varies from 8 to over 100 ft., but it is questionable whether any length less than 15 ft. should be employed in any pile foundation. In most cases the length ranges from 20 to 40 ft. The largest piles are used in dock construction or in bridge concrete-trestle work where the piles are located in deep water, the longest on record (1939) being 115 ft. (Art. 6-3).

It is quite general practice to construct precast piles without taper but pointed at the lower end. Piles without taper should always be used when conditions require the pile to act chiefly as a column. Where piles are tapered, the taper does not generally exceed $\frac{1}{4}$ in. per ft. and sometimes is not much over one-half of this. Quoting from the above noted manual:

> Piles with a uniform taper shall have a minimum lateral dimension of not less than 8 in. at the point and an average diameter of not less than 12 in. for lengths not over 40 ft. Piles over 40 ft. shall have a minimum taper of 1 in. in 8 ft. and a lateral dimension at the point of not less than 10 in.

The influence of taper on the bearing power of piles is discussed in Art. 6-18. Experience has shown the advantage of a point. In one job the substitution of pyramidal points for nonpointed piles increased threefold the number driven per day.

The use of metal shoes (see Fig. 6-3*b* for one form) is not of general application but may be desirable where the driving is hard or when boulders are present.

It is customary in good practice to fabricate the reinforcement as a unit, so that it can be easily handled and placed quickly in the form when the process of casting is underway. The reinforcement unit is held in correct position in the forms by suitable hangers and separators, so that the conditions assumed in designing the pile shall be realized in its construction.

If a pile is to have a hole in the center for the insertion of a jet pipe to be used in sinking, which may be more economical than to cast a jet pipe in the pile, either a tapered wooden core may be used or preferably a collapsible form; sometimes a thin metal tube is used instead and left in the pile. The objection to the solid core is that it requires occasional turning to prevent its sticking to the concrete when it is removed later.

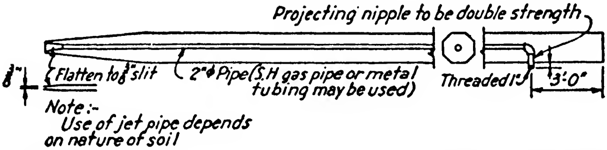


FIG. 6-4*a*.—Standard Jet Pipe for Concrete Piles. (*Courtesy of American Railway Engineering Association.*)

Figure 6-4*a* illustrates one standard form of jet pipe cast in the pile. It will be noted that the 2-in. pipe is flattened to a $\frac{3}{8}$-in. slit at the lower end.

The composition of the concrete is generally specified to give a compressive strength of 3,000 to 3,500 lb. per sq. in. The standard specifications of the American Railway Engineering Association call for a strength of 3,500 lb. per sq. in., using not more than 5 gal. of water per sack of cement. The maximum size of coarse aggregate is limited to 1 in.; but not more than three-quarters of the minimum clear spacing of reinforcement or the minimum distance from reinforcement to forms is allowed. It is further specified that the concrete mixture shall be of workable consistency dependent on the method of placement to ensure complete embedment of reinforcement and prevent honeycombing. It is specified that the concrete shall be kept wet during the curing period, which lasts until the concrete has attained a strength of 2,500 lb. per sq. in. as shown by test cylinders. Piles may not be handled or driven until the concrete has attained a strength of 3,500 lb. per sq. in. except that they may be handled if special provisions are made to keep the stresses down.

In manufacturing about 5,000 piles for a long concrete trestle across San Francisco Bay, the concrete mix used was 1:2.08:2.8,

with a water-cement ratio of 0.65. Tests showed compressive strengths of 3,000 and 4,300 lb. per sq. in. in 7 and 28 days, respectively.

Piles are cast in forms placed either horizontally or vertically, the former practice being the more common. When piles are cast in a horizontal position, special care should be exercised to provide an unyielding base so that the concrete may not be subjected to flexural stresses while in the process of setting. When the forms are vertical, special precautions must be observed in tamping or puddling the concrete to eliminate all voids. For either form of casting the use of vibrators is strongly recommended.

With horizontal molds the sides may be removed in from 24 to 48 hr., but the pile is allowed to rest on the base about a week longer, during which time it should be copiously showered with water to permit complete chemical action for the setting of the cement. By this time the concrete should have attained sufficient strength—at least 2,500 lb. per sq. in.—to permit the piles to be removed and piled in stacks to continue seasoning. They are usually allowed to harden for at least 3 weeks more before they are driven. The actual time to be allowed in each case depends on the temperature and humidity of the atmosphere, as well as on the character of the ground and the method of driving.

The hardening of the concrete may be materially hastened by curing with live steam under cover, by using a quick-hardening cement, or by using a specially rich mix. The piles for the docks of the Pennsylvania Lines at Cleveland (see Art. 6-3) were covered with canvas, and a steam-pipe line provided with outlet pipes was laid to discharge steam under the cover and to maintain a temperature of about 80°F. In building a bridge over the Illinois River at Florence, Ill., piles were driven 6 days after pouring. A strength of 3,765 lb. per sq. in. in 3 days was obtained by using a $1:1\frac{1}{2}:2$ mix with 3.75 gal. of water per sack of cement and mixing the concrete for 5 min.

Special devices are usually used to permit convenient handling of piles. A common detail consists of eyebolts or T-bolts screwed into plates cast in the piles near the center of the cross sections at points of lift, the bolts being well greased and turned slightly during the setting of the concrete to destroy the bond.

6-5. Designing and Handling Precast Piles. The steel reinforcement of a precast concrete pile is intended to resist the stresses due to (*a*) handling the pile, (*b*) driving the pile, and (*c*) the load which may come on it in its final position. The longitudinal bars

receive their maximum stresses when the pile is lifted from a horizontal position. Piles of moderate size are often picked up at or near the middle in going to and from the seasoning yard, or a line may be attached to one end to drag it to the pile driver. In the former case the pile must be strong enough to resist flexure due to its own weight; in the latter case the pile must not only sustain its own weight but also the impact due to meeting obstacles.

Some designers add 100 per cent to the weight of a pile to provide for the shock due to handling. This is probably excessive in cases where special provision is made for proper handling. The specifications of the American Association of State Highway Officials, which require the use of concrete that tests 3,000 lb. per sq. in. in 28 days, limits the stresses in the reinforcement due to handling to 12,000 lb. per sq. in., allowing 100 per cent of the calculated load for impact and shock. The American Railway Engineering Association specifications provide for a stress of 20,000 lb. per sq. in. in the reinforcement (intermediate grade of steel) and 25 per cent impact.

Handling stresses may be kept within almost any desired limits depending on the number of suspension points used. For large piles of record length as many as six suspension points may be employed, but in general three points of suspension will be enough. The reader is referred to a valuable article by E. G. Paulet entitled Spacing Suspension Points for Precast Concrete Piles, in *Engineering News-Record*, vol. 103, p. 260, Aug. 15, 1929, in which diagrams are presented for designing piles with one-point, two-point, and three-point suspensions.

For one-point suspension, the point of support will be at the center for piles without taper. If the weight of the pile is W lb. and the length l ft., the maximum moment will be $1.5Wl$ in.-lb. For two-point suspension the maximum moment will be least when the positive moment at the center equals the negative moments at the points of supports, which occurs when the points of supports are at a distance of $0.207l$ from the ends. The maximum moment is $0.257Wl$ in.-lb. For three-point suspension, where the rigging is such that the reactions are equal and one point is at the center, the other two points will be at a distance from the ends of $0.138l$ ft. for best results. With this arrangement the maximum negative moment will be at the first and third supports and the maximum positive moment will be at a distance of $l/3$ from the ends. These moments will be equal and of a value $0.1144Wl$ in.-lb. The moment at the center will be negative and will equal $0.0523Wl$ in.-lb.

Figure 6-5*a*, taken from the article just referred to, will be found helpful in designing square piles with equal tensile and compressive reinforcement. The curves are based on an allowable unit stress in the steel reinforcement of 16,000 lb. per sq. in. and in the concrete of 650 lb. per sq. in. By the use of the diagrams the

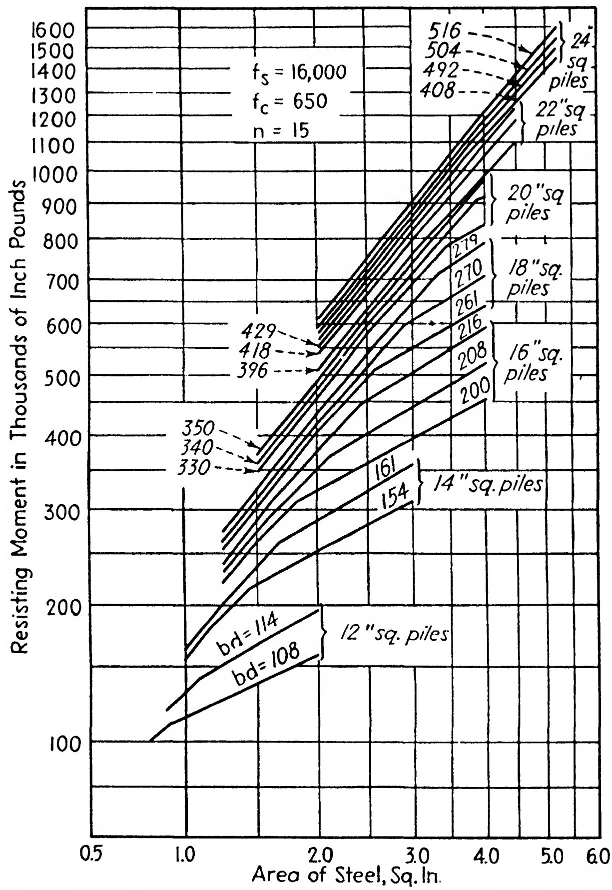


FIG. 6-5*a*.—Diagrams for Finding Required Reinforcing in Concrete Piles.

amount of longitudinal reinforcement at the tension face can be determined for any given bending moment for all commonly used sizes of square piles. In this diagram *b* is the width of the pile in inches and *d* the depth from the compression face to the center of the tension reinforcement.

For example, let it be required to determine the amount of reinforcement for an 18-in. pile, 60 ft. long, in which the longitudinal

bars have a 3-in. embedment. The concrete weighs 150 lb. per cu. ft. If a two-point suspension is used, the supports will be $0.207 \times 60 = 12.42$ ft. from the ends. The maximum moment is $0.257 \times 150 \times 1.5 \times 1.5 \times 60 \times 60 = 312{,}000$ in.-lb. If we begin in Fig. 6-5*a* at 312,000 in.-lb. and move horizontally to $bd = 18 \times 15 = 270$, the required area of steel at both top and bottom faces is 1.5 sq. in. Three $\frac{7}{8}$-in. round rods spaced 6 in. give an area of 1.8 sq. in.

The stresses set up in the reinforcement in driving piles are difficult to evaluate. Longitudinal bars are not economical elements in carrying the axial load that comes on the pile in its final position, since the unit stress cannot exceed n times the stress in the concrete, where n is the ratio or the modulus of elasticity of the steel and of the concrete. If this is 15 and the stress in the concrete is 650 lb. per sq. in., the stress in the steel will be 9,750 lb. per sq. in., as compared with a safe value of about twice this figure. In practice the percentage of steel in the sectional area of the pile ranges from about 0.6 to about 3 per cent. Good practice calls for a minimum of 1 per cent. The building codes of New York and Boston require a minimum of 2 per cent. The American Railway Engineering Association specifies that the cross-sectional area of the longitudinal reinforcement shall be not less than 1 per cent or more than 4 per cent of the average cross-sectional area of the pile.

The lateral reinforcement is of two distinct types. One consists of separate wire hoops either approximately square or circular in shape, spaced at intervals which vary more or less along the length of the pile; the other consists of continuous spiral wrapping which varies in pitch. Lateral reinforcement is intended primarily to increase the resistance of the concrete to longitudinal compression, but it also aids in resisting diagonal tension. The spacing of the lateral reinforcement at the two ends should be closer than at other points. The Building Code of New York City requires the use of lateral reinforcement consisting of $\frac{1}{4}$-in. round rods or wire as a minimum, with 12-in. spacing. The Manual of the American Railway Engineering Association specifies that "the lateral reinforcement shall consist of No. 7 steel wire, spaced not over 8 in. center to center throughout the length of the pile, except that for not less than 2 ft. at each end the spacing shall not exceed 3 in. center to center."

In general the sectional area of the concrete will be more than is necessary based on the load to be carried in its final position, especially if the pile is supported essentially by friction. The

stress in the concrete will usually not exceed 350 lb. per sq. in. For example, the Boston Building Code requires that the load on a pile having a sectional area of 169 sq. in. shall not exceed 30 tons, or 355 lb. per sq. in., ignoring the carrying capacity of the reinforcement.

6-6. Cast-in-place Piles. A cast-in-place pile is a concrete pile which is built in its permanent location in a hole prepared for that purpose. Although only some forms of premolded piles are protected by patents, all types of cast-in-place piles are the result of patents which relate more specifically to the method of construction for each type and the appliances used for that purpose. The hole may be formed by excavating with hand or power augers if the soil is stable and the depth not great, but it is more common to drive a casing.

Cast-in-place piles may be tapered piles in metal cases, cylindrical piles in metal cases, or uncased cylindrical piles. There are also a number of modifications of these fundamental types, such as the cylindrical type with a bulb at its base. The specifications of the American Railway Engineering Association require minimum diameters for cylindrical piles conforming to those given in Art. 6-4 for precast piles. Tapered piles are required to have a minimum diameter of 8 in. at the tip and to have a uniform taper of 1 in. in 4 ft. for shells up to 40 ft. in length and not less than 1 in. in 8 ft. for shells over 40 ft. in length. Piles between 40 and 56 ft. in length must have a minimum average diameter of 12 in.

Where the casing is thin, a collapsible steel mandrel or core is used in driving the pile, the mandrel being withdrawn prior to placing the concrete. The specifications referred to above require that the metal of shells driven with a mandrel shall have a thickness of not less than No. 18 B.W.G. ($\frac{3}{64}$ in.); those driven without a mandrel shall have a thickness of not less than No. 7 B.W.G. ($\frac{3}{16}$ in.); except where fluted shells are used, the minimum thickness may be No. 11 B.W.G. ($\frac{1}{8}$ in.).

Cylindrical piles are commonly placed by driving a heavy pipe and withdrawing the pipe as the concrete is placed. However, where ground conditions are not favorable casings may be used.

Concrete used is generally required to have a strength of 2,500 lb. per sq. in. and is placed continuously from tip to cutoff elevation.

6-7. Examples of Tapered Cast-in-place Piles. In making the type known as the "Raymond concrete pile," a tapering sheet-steel shell or casing is driven into the ground by means of a collapsible mandrel, as shown in the middle view of Fig. 6-7*a*. The shell of 20- to 24-in. gage sheet steel is made in conical sections about

8 ft. long. These sections are shipped knocked down and assembled to overlap tightly (in telescopic fashion) when in place. The shell tapers at the rate of 0.4-in. diameter per foot of length from point diameters of 8 or 11 in. to a maximum of 23-in. diameter at the head, in lengths up to 38 ft.

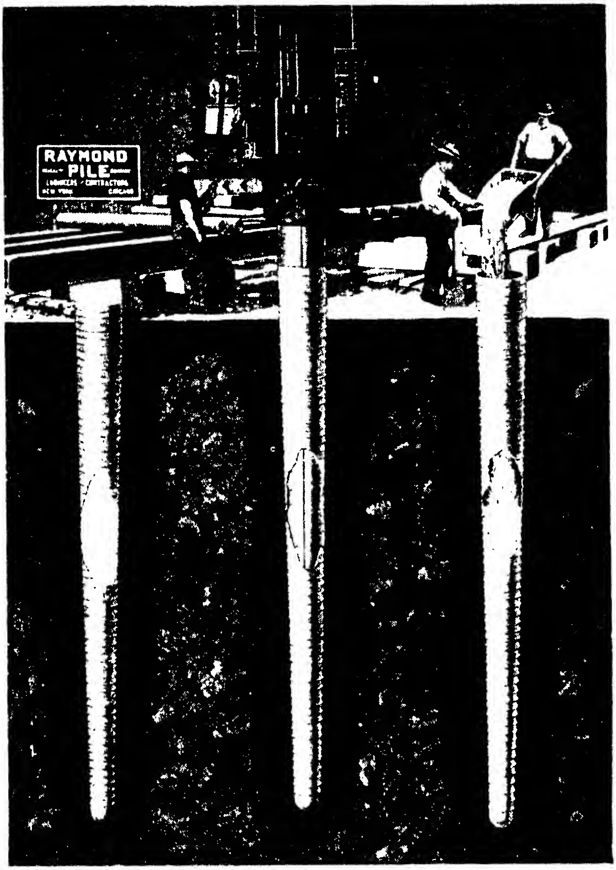


FIG. 6-7a.—Raymond Type of Cast-in-place Pile.

The object of the casing is to prevent earth and water from mixing with the concrete and to act as a mold that will preserve its shape after the mandrel has been withdrawn and until the concrete filling has hardened. The shell is reinforced with $\frac{1}{4}$- or $\frac{3}{8}$-in. wire spiral (see Fig. 6-7a) spaced about 3 in. to further stiffen the casing. Before placing the concrete, the interior of the shell can be inspected by means of an electric light, by light reflected from a mirror, or by the light reflected from the surface of water thrown into the casing.

For lengths in excess of 38 ft. the company which developed this pile uses a type known as the Raymond step-tapered pile, which generally tapers 1 in. in each 8 ft. of length in a series of steps from a point diameter of 8 in. or more. This pile may be made to lengths of 100 ft. or more, the record length being 115 ft. The 8-ft. cylindrical shells are made of spirally corrugated metal and have a plow ring welded to the lower end. To the lower projecting fin of this plow ring there is welded a short spirally corrugated inner sleeve to permit the lower section to be screw-connected to the section above. As shown in Fig. 6-7*b* the heavy hollow core used to drive the pile has a bevel-stepped outer surface to bear directly on the beveled surface of the plow ring.

FIG. 6-7*b*.—Raymond Step-tapered Concrete Pile Showing Shell and Driving Core.

On account of the heavy weight of the cores used in driving Raymond piles, the pile driver is of heavy construction. Figure 6-7*c* shows two machines in action, one driving a vertical pile and the other a batter pile.

Where no mandrel is used, the shell must be strong enough to withstand the driving stresses. The Monotube pile is one example of this class of construction. The shell is formed of cold-rolled fluted steel, the lightest gage used being No. 11 and the heaviest No. 3. The standard tapers are 1 in. in 7 ft. and 1 in. in 4 ft. Piles are also available without taper. Any length desired is available by combining a number of sections, these sections usually being in 20-ft. lengths with welded connections. A steel point is welded to the tip and a driving collar to the butt.

6-8. Examples of Uncased Cylindrical Piles. The Simplex pile, introduced in 1903, is made by driving a steel pipe—usually 16 in. in diameter—with a special metal base and then filling the hole with concrete as the pipe is gradually withdrawn. The pipe must be extra heavy and at least as long as the pile to be formed, and the pile driver must have extra strength equipment to pull out the pipe. The pressed-steel base is slightly concave downward, and the rim is lapped a short distance upward over the outside of the pipe. This base remains in place, and hence a new one is needed for each pile.

A ram may be employed to force each batch of concrete into place against the surrounding earth until the hole is completely

filled. This increases the diameter of the pile somewhat beyond that of the pipe.

The pedestal pile, invented by Hunley Abbott, may be regarded as a modification of the Simplex pile by the addition of a bulb-shaped base or pedestal at the foot. Its form is intended to take a larger measure of advantage of a lower stratum of higher bearing

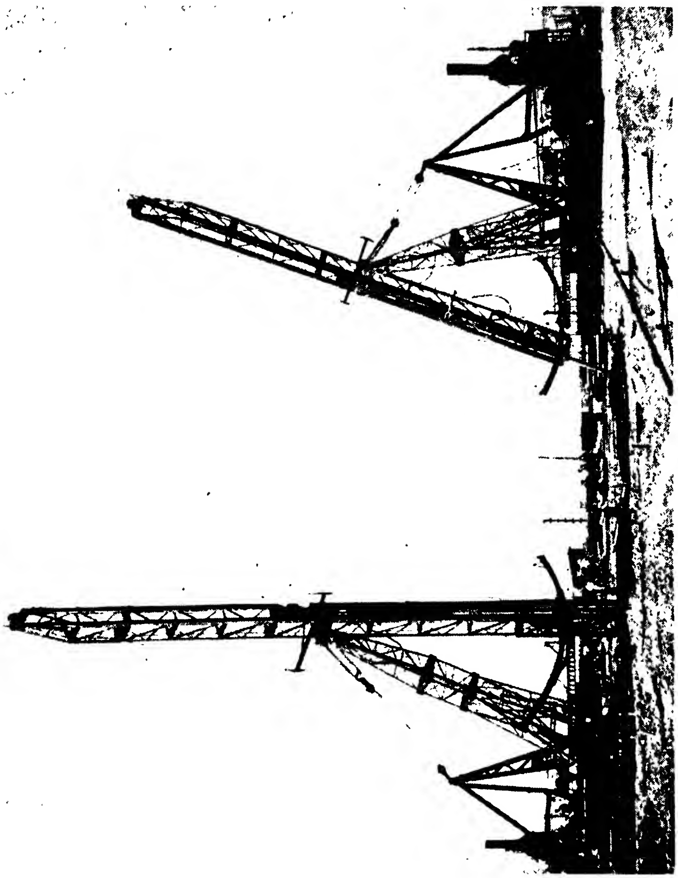


FIG. 6-7c.—Steel Pile Drivers Used in Driving Casings of Raymond Concrete Piles.

capacity. By thus increasing its bearing area at the foot, its action simulates that of metallic disk and screw piles (Art. 7-10).

Figure 6-8*a* illustrates the placing of the MacArthur pedestal type for lengths up to 40 ft. A core and casing assembly is first driven into the ground, the core being solid steel and the casing a pipe with a shell thickness of about $\frac{1}{2}$ in. In the second step the core is removed and a charge of concrete is dropped to the bottom

of the casing. The casing is then pulled up $1\frac{1}{2}$ to 3 ft. with the weight of the core and hammer, perhaps 7 tons, resting on the concrete. The fourth step is to ram out the concrete, after which the core is removed and the casing filled with concrete. The fifth step is to withdraw the casing while the concrete is under pressure from the weight of the core and hammer.

In driving through incompressible clay where pronounced upheaval of the soil occurs, resulting in damage to piles already in place, an open casing may be driven without the core. On reaching the desired penetration, the casing, filled with soil, is pulled up and the filling removed. The empty casing is then redriven and the concrete filling placed.

The value of this method was demonstrated in placing concrete piles near Catskill, N. Y. Borings had indicated rock at depths

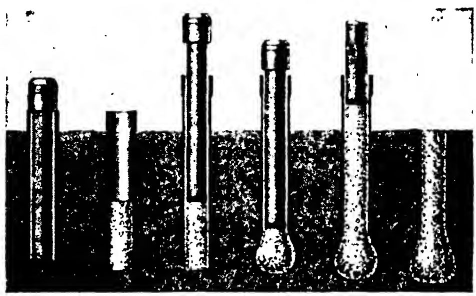


Fig. 6-8*a*.—MacArthur Pedestal-type Pile.

varying from 15 to 33 ft. below the pile cutoff, the overlying material being a comparatively soft but impermeable clay made up of thin layers with water between. The contract was let on the basis of using shell-less cast-in-place piles placed by the usual method. About 70 of these piles were driven, when it was discovered that the tops of the piles were rising several inches as additional piles were driven. Investigation showed that there were horizontal breaks occurring, and it was concluded that the uplift was in the upper part of the piles only. A change was therefore made to a pile with a corrugated metal casing. In placing these piles a pedestal of concrete was driven out at the bottom with the expectation that this would hold the pile down. However, after driving 50 piles of this type, it was found that they also were coming up, indicating that the upheaval probably extended the entire length of the pile. The problem was solved by using the cored-out pipe pile as described in the preceding paragraph.

6-9. The Franki Pile. Another type of cast-in-place pile, which was developed in Europe, is the Franki pile, which has a pedestal base and corrugated stem. In making the pile a pipe casing is held vertically, its lower end resting on the surface of the ground at the point where the pile is to be driven. A charge of dry concrete is then placed in the bottom of the casing, after which a drop-hammer weighing from 4,000 to 6,000 lb. delivers blows to the concrete through a fall of several feet. This forces the concrete plug into the ground, the side friction of the concrete on the pipe pulling the casing down with the concrete. On completion of sinking, the casing is raised slightly and held in place by cables. The hammer is again applied to the concrete, forcing it downward and outward to form the enlarged base. The shaft is formed by introducing charges of concrete, each charge being rammed while the casing is withdrawn a short distance, this process resulting in a series of corrugations in the shaft.

6-10. Precautions against Damage. Since precast piles cannot be driven until they are sufficiently seasoned, they may be placed in any order in the required foundation. This cannot be safely done with cast-in-place piles. When the core or the pipe is driven for a given pile, it displaces and compresses the earth adjacent to the hole which is formed, and the elastic earth tends to relieve its stress by crowding back. Even if the shell, which is left in the hole, or the weight of the concrete, when no shell is employed, is able to resist the outside pressure until the cement is set, it is very probable that the green concrete will be damaged by the vibration and increased earth pressure due to driving additional piles, after the setting of the cement has progressed to a certain extent and before its completion. The specifications of the American Railway Engineering Association require that no cast-in-place pile shall be filled with concrete until all adjacent piles within a radius of $4\frac{1}{2}$ times the average pile diameter, or not less than 5 ft., have been driven to the required resistance. After a shell has been filled with concrete, no shell or pile shall be driven within 15 ft. thereof until at least 4 days have elapsed.

Numerous examples are available to demonstrate that great care must be used in placing uncased piles. In one example, failure was due to the fluid alluvial soil penetrating between batches of concrete, thus separating the piles into sections about 5 ft. long. In another, the cement failed to set on account of certain chemical constituents in the ground water, ascertained later by analysis. In still other cases, piles had their section areas reduced from 20 to 100

per cent, and were bent out of line. The liability of green concrete to suffer damage by driving adjacent piles is increased when hard strata alternate with soft ones. Unless protected by a shell there is more or less danger of some of the cement being washed out by underground flowing water, or, on the other hand, that the cement may be deprived of some of the water, which it needs to set completely, by the absorbent earth. In the early years, proper precautions were sometimes not taken to prevent pulling up of concrete as the casing was lifted, which resulted in poorly formed piles.

The difficulties sometimes encountered in cast-in-place pile construction are well exemplified in the foundation work of the Internal Revenue Building and the Department of Commerce Building in Washington, where 21,500 piles were driven.[1] For a distance of from 8 to 10 ft. below the basement floors the soil was swampy muck, below which there was a stratum of nonwater-bearing compact hardpan and dense gravel, varying in thickness from 1 to 3 ft. Under this hardpan there was a considerable thickness of fine soft clay which rested on water-bearing gravel and sand. This gravel and sand stratum proved to be a water basin with the water sealed in tight by the overlying hardpan under a hydrostatic head sufficient to force the water to the ground surface wherever the seal was broken.

Both foundations consisted of concrete piles cast in place by driving thin steel shells and filling them with concrete. In order to guard against undetected shell failures, it was specified that no shell was to be filled with concrete until all adjacent shells within a radius of 5 ft. center to center had been driven to required bearing.

A great deal of trouble was experienced by the movement of the soil and by the presence of the water. Where the upper stratum of hardpan and dense gravel was thin, the displaced material was absorbed in the soft muck or clay as the driving progressed; but, where the thickness approached 3 ft., the pressure of the hardpan and dense gravel caused by the displacement of the material frequently crushed near-by shells. In some cases the shell being driven would collapse immediately on withdrawal of the mandrel. In other cases failure would come when adjacent piles were being driven. Shells driven into the lower gravel would frequently be cut or torn open at or near the point. This was of serious moment because any opening immediately started a flow of water up through the pile form, making it impossible to place concrete until the difficulty was overcome. In most cases where a shell collapsed or was torn

[1] See *Eng. News-Record*, vol. 102, p. 823, May 23, 1929.

open, a new shell was driven inside the old one. In some cases it was necessary to abandon a pile and drive a replacement one close by. The total number of shells used for the Internal Revenue Building and the Department of Commerce Building were 32 and 49 per cent, respectively, in excess of the number of piles specified on the plans.

6-11. Hollow Precast Piles. A method of placing concrete piles which has some of the characteristics of both the precast and the cast-in-place types involves the driving of a precast reinforced-concrete shell, which is later filled with a lean mix of concrete. This method reduces the weight of the pile to be handled and also makes possible a shell of hard, strong, and durable concrete with lower grade concrete for the core. As an example, in building the foundations of an ocean pier at Long Branch, N. J., hollow piles, 24 in. square on the outside and 13 in. square on the inside, were driven and later filled with concrete. The lengths ranged from 45 to 68 ft. The reinforcement consisted of $1\frac{1}{4}$-in. round rods tied together at intervals with $\frac{1}{4}$-in. wire collars.

Another example, is the Peerless concrete pile. It has a sectional reinforced-concrete shell, which is driven down together with a steel driving pipe, both of which bear on a pointed cast-iron shoe, the latter being left in the ground. After the steel pipe is withdrawn, the shell is first inspected and is then filled with concrete by a special tremie designed for the purpose. The use of the steel pipe protects the concrete shell from severe stresses during driving.

Hollow concrete piles, 39 in. in diameter and 149 ft. long, were used for the substructure of a bridge in Stockholm harbor, the piers consisting of reinforced-concrete caps, carried by groups of piles in a manner similar to timber-trestle construction. The pile shells were 3 in. thick and well reinforced. The concrete was a $1:1\frac{3}{4}:1\frac{3}{4}$ mix, with a compressive strength of 3,500 lb. per sq. in. in 28 days. After driving the piles, they were cleaned out with an air ejector and filled with concrete. These piles might be classified as open-cylinder caissons (see Art. 9-8), but, as they were placed by driving, piling is probably a better classification. Hollow concrete piles 4 ft. in diameter and 200 ft. long have been used in Europe.

6-12. Concrete Piles in Sea Water. Where concrete piles in sea water extend above ground surface, there is the same general tendency for the concrete to disintegrate that is typical of a great deal of concrete placed in sea water. Much study has been given this problem, but we do not yet know the real reasons for the deterioration of the concrete. The causes are probably both

mechanical and chemical. Mechanical action includes the abrasive effect of debris, ice, wind, and waves. In a cold climate the water, entering the pores, interstices, and fine cracks of the concrete within and above the tidal range, freezes and by its consequent expansion bursts the confining passages and destroys the concrete.

The causes of chemical disintegration are not well understood, but it is known that the calcium hydroxide which forms during the hardening of the cement is soluble in water and so if water seeps into the concrete the calcium hydroxide will be dissolved and carried in solution out to the surface of the concrete. Another factor is the interaction of the magnesium sulfate contained in the sea water and the calcium hydrate, or so-called "free lime," of the concrete. Among the products of this reaction is calcium sulfate, whose molecule is slightly larger than the original calcium-hydrate molecule; hence a swelling effect is developed to promote disintegration. To date, little success has attended efforts to make a cement containing nonsoluble chemical combinations free from attack by the compounds existing in sea water.

As deterioration of the concrete progresses, the reinforcing steel in the concrete becomes exposed and rusting begins. This rust has many times the volume of the original steel and so another agency is set up to promote cracking of the concrete.

It appears that the best way of preventing deterioration of the concrete is to keep the water out. Among the methods that have been used to accomplish this are the following: (*a*) applying paint to the surface of the concrete, (*b*) using integral waterproofing in the concrete, (*c*) impregnating the concrete with asphalt, and (*d*) placing a bituminized concrete casing around the pile.

Numerous tests have shown that a thin coating of paint or preservative on the surface is effective for only a short time.

The theory of integral waterproofing is that by the addition of an admixture, such as lime, clay, diatomaceous earth, pozzuolanas, emulsions, etc., the density and impermeability of the concrete can be increased. However, tests indicate that better results can be obtained by substituting additional cement for these admixtures and by keeping the water-cement ratio down to a minimum.

6-13. Asphalt-impregnated Piles. The idea of impregnating a porous lean-mix concrete pile with asphalt to form what was called Duocrete was developed in 1920 by G. F. Nicholson, Engineer for the Los Angeles Harbor Department. Impregnation was secured by immersing the concrete pile in a bath of hot asphalt, at a temperature of from 450 to 500°F. for a period of from 15 to 24 hr.

The impregnation results were satisfactory, but the combination of a high destructive temperature and a lean-mix concrete resulted in a low-strength pile, which proved unsatisfactory under service conditions.

This resulted in adopting a vacuum-pressure treatment similar to that used in creosoting timber (Art. 4-18). A dual-mixture concrete is used, that for the core of the pile being a dense, high-strength concrete developing a compressive strength of from 4,000 to 6,000 lb. per sq. in.; while the concrete comprising the $1\frac{1}{2}$-in. thick shell, which extends from the top of the pile to at least 3 ft. below low tide, while strong, is much more porous. The maximum temperature used is 250°F., a figure well below that which would damage the concrete.

The core concrete approximates a 1:5 mix, the fine and coarse aggregates being measured separately. The maximum quantity of water is limited to 52 lb. per cu. ft. of cement, and the slump is limited to 2 in. The jacket concrete has a maximum size of aggregate of $\frac{3}{4}$ in. and approximates a 1:6 mix, with only sufficient water to thoroughly moisten the mixture, 42 lb. per cu. ft. of cement being a maximum. The core and jacket are cast simultaneously by the use of vertical steel-plate separators which are withdrawn as the placing of the concrete proceeds. Air hammers are used to vibrate the forms in order to secure maximum density of the concrete.

After curing for at least 60 days, the piles are placed in a dry-kiln chamber and the temperature is raised gradually for about 16 hr. to 250°F. After being subjected to this maximum temperature for from 2 to 4 hr., they are quickly transferred into the main preheated treating cylinder and further dehydrated under a 26- to 28-in. vacuum. Asphalt is then admitted into the treating chamber at a temperature of about 250°F., the vacuum being maintained throughout its introduction. After the chamber has been filled with asphalt, an air pressure of from 150 to 175 lb. per sq. in. is introduced and maintained for from 12 to 14 hr., during which time the temperature is gradually lowered to 200°F. When the asphalt is withdrawn, the piles are allowed to cool gradually either in the pressure cylinder or in a special cooling kiln.

The final impregnation of this method amounts to about $1\frac{1}{2}$ in. in the portion of the pile having the drier mixed shell and from $\frac{1}{4}$ to $\frac{3}{8}$ in. in the remainder. The cost of impregnated piles is from 25 to 40 per cent in excess of that of ordinary concrete piles. For an excellent article on this type of pile the reader is referred to *Civil Engineering*, vol. 1, p. 1243, November, 1931.

The impregnation method of waterproofing concrete requires a very large plant for long piles, and the impregnation cannot be limited to that part of the pile where it is needed. To simplify the protection procedure and to reduce the cost, slabs of precast asphalt-penetrated concrete may be cast peripherally into the pile for that part of its length affected by the rise and fall of tide.

Figure 6-13*a* illustrates such a type of pile as used for a concrete-trestle bridge across the Bellamy River in New Hampshire, where the piles varied in length from 70 to 105 ft. The view is through the armored part of the pile, but the unarmored part has the same dimensions. All piles were tapered for 6 ft. at the bottom to a 10-in. square point. The casing zone was 18 ft. long, which covered the 6-ft. tidal range with ample margins above and below. The pile concrete was a $1:1\frac{1}{2}:2\frac{1}{2}$ mix, the maximum size of aggregate 1 in., the slump 2 in., and the water-cement ratio $4\frac{3}{4}$ gal. of water per sack of cement. A variation of 15 per cent in the ratio of fine to coarse aggregate was allowed to secure workability. The concrete of the bituminized encasement was a $1:3\frac{3}{4}:1\frac{1}{4}$ mix, the maximum size of aggregate $\frac{3}{8}$ in., the slump 2 in., and the water-cement ratio 4 gal. per sack of cement.

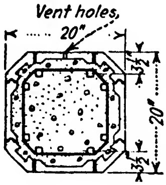


FIG. 6-13*a*.—Impregnated Concrete Slabs Encasing Concrete Pile.

The peripheral covering consisted of eight precast asphalt-impregnated concrete slabs in sections 3 ft. long, recessed on the inside to bond with the pile concrete. The method of construction was to first place six face slabs on the bottom board of the form and then pour hot asphalt in the butting joints. The lower corner slabs were next placed and the joints sealed. Following this the pile-form sides were erected, the side face slabs placed and asphalted, and the upper corner slabs were placed. The concrete was then poured and compacted with internal vibrators, after which the top slabs were placed.

6-14. Composite Types. The cheapness of wood piles has been combined with the durability of concrete to form a number of combinations of the two. One combination consists of first driving a timber pile to a depth such that the top of the pile will be below permanent ground-water level and then superimposing a concrete pile on top.

One method of construction is to first drive a mandrel fitted into a heavy steel-pipe casing—similar to the Simplex and MacArthur concrete pile procedure—until the bottom of the casing is well below ground-water level. The core is then withdrawn from the casing

and a wood pile, with an accurately formed tenon about 18 in. high and $9\frac{1}{2}$ in. in diameter, is inserted in the casing. The tenon of the pile fits into a socket in the lower part of the mandrel so that, in driving, the mandrel bears on the top of the tenon and on the shoulder of the pile. If the mandrel is used as a follower, the wood pile is driven through the casing until the head is below the ground-water level and a satisfactory resistance is developed. The mandrel is then removed, and a corrugated-metal shell is lowered into the casing to surround the tenon. This shell is filled with concrete, after which the casing is taken out.

The Raymond composite pile is placed by first driving a wood pile to ground level and fitting on the top of the wood pile a collapsible mandrel encased in a spirally reinforced, sheet-steel shell. The combined unit is then driven to its final penetration, when the mandrel is withdrawn, leaving the shell to serve as a mold for the concrete.

To secure a satisfactory composite pile care must be exercised (*a*) to maintain perfect alignment between the wood and concrete sections; (*b*) to protect the head of the wood pile against injury in driving; (*c*) to provide a tight joint which will exclude sand, mud, and water; and (*d*) to provide enough reinforcement and anchorage to lock the wood and concrete sections together so that the joint will have sufficient transverse strength and resistance to ground-pressure upheaval.

The tenon is sometimes reinforced by a wrapping of spirally wound, galvanized wire. It is extremely difficult to maintain a watertight joint between the metal shell and the shoulder of the wood section. In one type of joint the corrugated metal shell rests on a pan having an upstanding flange. This pan bears on the shoulder of the wood pile, a hole in the center of the pan making it possible to slide the same down over the tenon. In the Raymond design a hollow boot encases the lower part of shell and mandrel and at the same time fits tightly against the lower part of the vertical surface of the tenon. To develop a strong joint, a mesh of reinforcing bars is often placed in the concrete surrounding the tenon and extending some distance above. To provide resistance against uplift, a horizontal bar or short length of pipe may be placed through a hole bored in the tenon, this bar extending out into the concrete. Another device consists of a vertical reinforcing bar screwed into a steel socket embedded in the center of the tenon, the socket being held in place by a horizontal pin extending through the tenon and socket. The reinforcing bar extends well up into the concrete.

Another type of composite pile consists of a wood core and concrete shell. An example of this is the Ripley combination pile, shown in Fig. 6-14*a*. The reinforcement consists of wire mesh wound spirally with the concrete around the pile, to which it is attached by staples, the final lap being tied with wire. Before concreting, spikes are driven into the timber at intervals on its surface. The concrete is a 1:2:3 mixture. Another combination pile is made by shooting a 2-in. thickness of Shotcrete around timber piles as noted in Art. 4-19. The Shotcrete is usually placed in two layers, with reinforcement consisting of longitudinal bars and wire mesh. In placing concrete or Shotcrete, care must be taken to

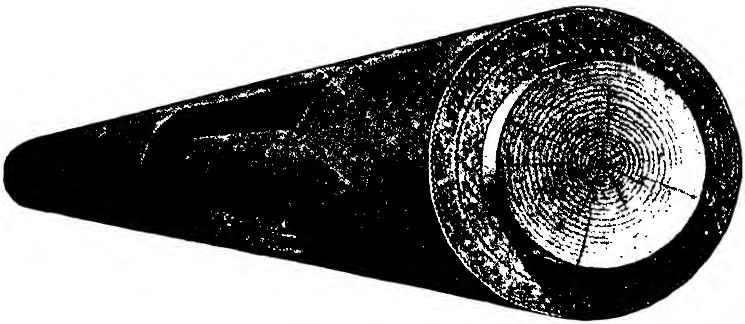


FIG. 6-14*a*.—Ripley Combination Pile.

have the wood thoroughly water-soaked, else the subsequent swelling will break the protection coating.

6-15. Drivers, Hammers, and Caps. To drive precast concrete piles, or the casings for cast-in-place piles, the pile driver and its equipment have to be much stronger and more rigid than required for timber piles on account of the greater weight to be handled and the heavier hammers required. A 50-ft. timber pile may weigh about 2,500 lb., whereas a concrete pile of the same length will weigh perhaps 12,000 lb. Concrete piles weighing from 2 to 4 tons are very common, and those of 6 to 8 tons are employed in heavy construction, while the record weight is 33½ tons for 24- by 24-in. piles, 115 ft. long, as noted in Art. 6-3. Steel pile drivers are widely used, as they are stiffer, more durable, and lighter for the same strength than those built of wood. Figure 6-7*c* illustrates a skid type of steel pile driver as used by the Raymond Concrete Pile Company.

In selecting a hammer, choose the heaviest one possible that will not damage the concrete in driving a properly cushioned pile. Attention should be given to both the foot-pounds of energy per

blow and to the weight of the striking parts. The committee referred to in Art. 4-3 recommended that the weight of the striking part should not be less than 2,500 lb. The energy per blow should not be less than 3,750 ft.-lb. per cu. yd. of concrete in the pile. As noted in Art. 3-8, the heaviest steam-hammer now listed in manufacturers' catalogues weighs 39,050 lb., develops 50,200 ft.-lb. per blow, and has a weight of striking parts of 20,000 lb.

It is desirable that the weight of the striking parts be adequate. If too large a proportion of the energy is in the form of velocity, we have a condition analogous to the action of a light drop-hammer with a large fall as explained in Art. 4-1, where a large portion of the energy is dissipated in destructive work. Another penalty paid for a high ratio of energy to weight of striking parts is the lifting action exerted by the steam on the nonmoving parts of the hammer. This tends to reduce the advantage gained by the dead weight of the hammer resting on the pile. Some specifications limit the velocity of the hammer to that corresponding to a free fall of 5 or 6 ft. Drop-hammers are rarely used in driving concrete piles. Where used, the weight of the hammer should not be less than that of the pile and the fall should not exceed 8 ft. Very satisfactory results were secured on some building foundations in Pittsburgh by using drop-hammers weighing from 7,000 to 12,000 lb. each. These hammers were handled by three-part crucible-steel lines roved at the lower end over sheaves set in the hammer castings. The fall of the largest hammer was limited to about 8 ft. but was usually less.

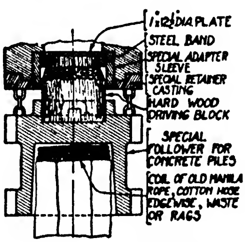


FIG. 6-15*a*.—Cap Used on Industrial Brownhoist Steam-hammer for Driving Concrete Piles.

Wherever possible, concrete piles should be driven with the aid of the water-jet so that the duty of the hammer becomes secondary.

The successful driving of precast piles without injury to the pile depends largely on the effectiveness of the driving cap used to cushion the blows. As explained in Art. 3-11 most steam-hammers are designed so that the hammer blow strikes on an anvil block rather than on the pile directly. This somewhat lessens the impact of the blow, but in the case of concrete piles further cushioning is effected either by the use of caps regularly furnished with the hammer or by the use of specially designed caps. These caps or driving heads are also necessary in order to adapt the base of the hammer to the different shapes of piles.

The driving head of the Warrington-Vulcan hammer is a steel casting shaped to engage the leads of the driver, with a ring at the

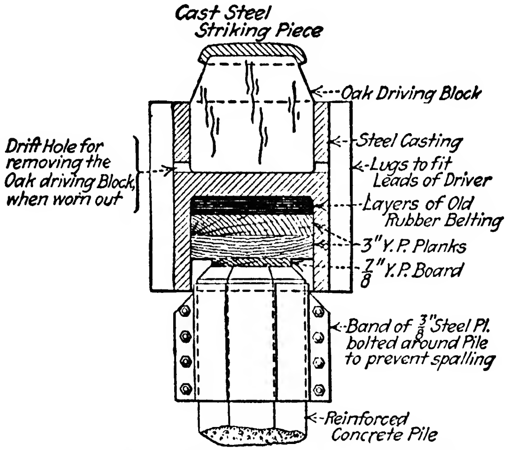


FIG. 6-15b.—Cap for Driving Concrete Piles.

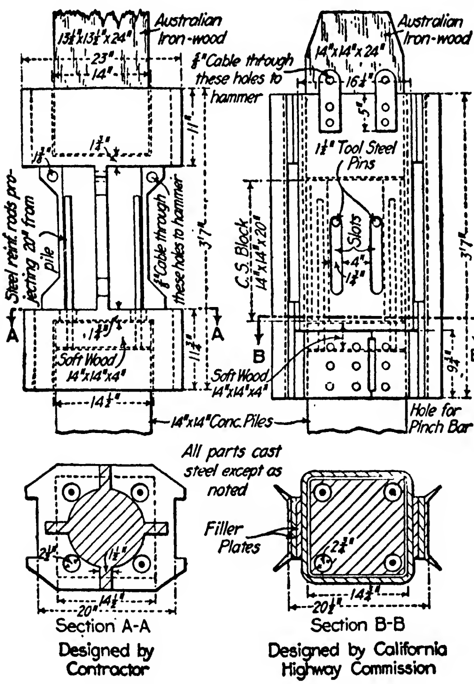


FIG. 6-15c.—Pile Caps Used by California Highway Commission.

top filled with a short wooden block to receive the blow of the hammer. The lower surface is shaped to fit the pile top and is

recessed sufficiently deep to allow for the placing of a layer of wooden planking between the pile and driving head.

Figure 6-15*a* illustrates the type of cap, or follower as it is called, used on the Industrial Brownhoist hammer. This cap is attached to the hammer by short lengths of chain. A hardwood driving block, with a steel band at the top, is fitted into the top of the cap, and the cap rests on the concrete pile, with a cushion of manila rope, cotton hose, or rags between the cap and the pile.

Figure 6-15*b* illustrates a specially designed cap used in driving the foundation piles of the Municipal bridge approach at St. Louis. The construction is fully explained in the diagram. Figure 6-15*c* illustrates two types of caps used very successfully where reinforcing rods extend up from the pile; Fig. 6-3*d* shows a cushion consisting of a 4-ft. thickness of timber.

6-16. Formulas for Bearing Power. The pile-driving formulas appearing in Art. 5-4 are applicable to concrete piles as well as to timber piles. However, in driving concrete piles, the weight of the drop-hammer or the weight of the ram of a steam-hammer bears a much lower ratio to the weight of pile plus cap than in the case of timber piles. Consequently a formula like the *Engineering News* formula (Eq. 5-4*c*), which has no term depending on this ratio, may be quite satisfactory when applied to timber piles and wholly unsatisfactory when applied to concrete piles. As explained in Art. 4-1, for low values of $r = W_h/W_p$ a large percentage of the hammer energy is lost in heat development, with the result that the penetration of the pile is small, this giving a false picture of the pile resistance. The modified *Engineering News* formula (Eq. 5-4*d*) was developed to correct, at least in part, this defect in the original formula. The Hiley or the Boston Building Code formula is recommended where heavy concrete piles are involved.

Tests were made in 1932 by the Missouri Pacific Railroad on two concrete piles, as well as on the timber pile noted in Art. 5-4. The data for the first concrete pile is as follows: W_h = 5,000 lb. (single-acting steam-hammer); W_p = 5,600 lb.; r = 5,000/5,600 = 0.893; h = 3 ft.; s = 0.6 in.; L = 240 in.; and A = 212 sq. in. The value of E will be taken as 2,000,000 lb. per sq. in. The static load test showed a value of 122,000 lb. with a permanent settlement of 0.60 in.

In applying the Hiley formula, e = 0.9 (Table 5-3*a*); n = 0.25 (Table 5-3*b*). To find the value of C from Table 5-3*c*, no information being available, we shall assume that an effective driving cap was used and that R_a will be about 37,000 lb. Hence $R_d = 3 \times$

37,000 = 111,000 lb., and $R_d/A = 111{,}000/212 = 525$ lb. per sq. in.; hence from Table 5-3*c* we have $C = 0.29$. Therefore Hiley (Eq. 5-4*a*):

$$R_a = \frac{4 \times 0.9 \times 5{,}000 \times 3}{0.60 + \dfrac{0.29}{2}} \; \frac{0.893 + (0.25)^2}{0.893 + 1}$$
$$= 36{,}500 \text{ lb.}$$

In the Boston Code formula (Eq. 5-4*b*), assuming

$$R_a = 34{,}000, \qquad K = \frac{1.5 \times 34{,}000 \times 240}{212 \times 2{,}000{,}000} + 0.05 = 0.079, \text{ hence}$$

Boston Code (Eq. 5-4*b*):

$$R_a = \frac{3.6 \times 5{,}000 \times 3 \times 0.9}{0.60 + 0.079} \; \frac{0.893}{0.893 + 1} = 33{,}800 \text{ lb.}$$

Engineering News (Eq. 5-4*c*):

$$R_a = \frac{2 \times 5{,}000 \times 3 \times 0.9}{0.60 + 0.1} = 38{,}600 \text{ lb.}$$

Modified *Engineering News* (Eq. 5-4*d*):

$$R_a = \frac{2 \times 5{,}000 \times 3 \times 0.9}{0.60 + \dfrac{0.1}{0.893}} = 37{,}900 \text{ lb.}$$

The factor of safety based on the Hiley formula is 122,000/36,500 = 3.34.

In the second pile test: $W_h = 7{,}500$ lb. (single-acting steam-hammer); $W_p = 15{,}000$ lb.; $r = 7{,}500/15{,}000 = 0.50$; $H = 3.25$ ft.; $s = 0.429$ in.; $L = 240$ in.; and $A = 480$ sq. in. The static load test showed a value of 188,000 lb. with a permanent settlement of 0.636 in. Assuming that $C = 0.23$, the Hiley formula gives $R_a = 60{,}500$ lb. The results by the other formulas are as follows: Boston Building Code, 52,700 lb.; *Engineering News*, 83,000 lb.; and Modified *Engineering News*, 69,800 lb.

The factor of safety based on the Hiley formula is 188,000/60,500 = 3.11. It will be noted that for this low value of r the value of safe bearing load by the *Engineering News* formula is much higher than the values given by the others.

6-17. Choice of Type. Where piles are used to support a structure above open water as in pile trestles, wharves, piers, etc., they are required to resist flexure, as well as to act as columns. Precast

piles are especially adapted for this service, and they should be molded without taper, at least for that part of the length that is not in the ground.

Where a pile is driven through soft soil to a hard stratum, so that it must act as a column, it must be well reinforced; hence the precast pile is the logical type to use. However, if a reinforced cast-in-place pile type is adopted, it should be one that leaves a casing in the ground which can retain its form until the concrete has hardened. The pile should be without taper so as to have a large bearing area in the harder substratum.

If, however, the overlying material is firm and stable, then those types of cast-in-place piles in which the pipe is gradually withdrawn may be used. If the underlying stratum which is to bear a considerable part of the load is not sharply defined on its upper surface, it may be desirable to increase the bearing surface of the pile by means of an enlarged base.

Where the ground is compressible at the top, but not soft, and gradually increases in density downward, any one of a number of different types may be employed. But all of them should be without taper so that proper advantage can be taken of the greater bearing at the foot and the greater frictional resistance at the lower surface of the pile.

If the ground consists of silt or alluvium for a great depth and increases but slowly in density with the depth, so that its bearing power depends practically all on skin friction, the choice between a tapered and an untapered pile depends on two factors. The pile with a constant section has a slightly larger superficial area for a given volume, the greatest difference being practically less than 5 per cent. Such a pile has the additional advantage of having a larger proportion of its surface in the lower part of the pile, where the friction is slightly greater. But the tapered pile has a larger section area of concrete at the top to transmit the load, and that larger area may govern in some cases. As the load is gradually transferred to the surrounding earth in passing downward through the pile, the decreasing section area of a tapered pile makes it conform more closely to one of uniform strength throughout.

Sometimes deep beds of clay require pile foundations because the upper stratum becomes soft during the flood season, whereas during the most favorable time for construction, the clay is too hard to drive piles. Under such conditions the holes may be prebored.

6-18. Effect of Taper. To indicate the relative properties of tapered and straight concrete piles, the following example may be

considered: Let the tapered piles be 20 ft. long and the diameters of the head and foot be 20 and 6 in., respectively, making the volume 20.2 cu. ft. Let a straight pile be taken having the same length and volume; its diameter is therefore 13.6 in. In the tapered pile 44.5 per cent of its volume is in the uppermost quarter of the pile and 74.2 per cent in its upper half, while 35.1 per cent of its available surface for frictional resistance is in the top quarter and 63.5 per cent in the upper half of the pile. Since piles are frequently spaced 3 ft. between centers, let it be assumed that the compression of the earth surrounding a pile, which diminishes from the pile outward according to some law depending upon the nature of the material, be equivalent to a uniform compression, limited to a radius of 1.5 ft. from the center of the pile. Dividing the depth into four quarters, the ratio of the displacement of the pile to the corresponding volume of the compressed earth is, accordingly, 25.8 per cent for the top division, 16.9 and 9.8 per cent for the next two divisions, and 4.7 per cent for the lowest division. For the straight pile the corresponding values are 14.3 per cent for each division. The proportions of the total frictional area of the tapered pile are 35.1, 28.4, 21.6, and 14.9 per cent in the four divisions, respectively, while those for the straight pile are each 25.0 per cent. The frictional areas of the tapered and straight piles are 68.1 and 71.2 sq. ft., the difference being a little less than 5 per cent. It should be noted especially that about 45 per cent of the total equivalent compression of the earth was expended in the top division and very nearly 75 per cent in the upper half of the depth.

It may be considered objectionable to adopt a large taper for the following reasons: the compression of the earth is thereby made a maximum near the surface and a minimum near the foot of the pile, which is contrary to the fundamental principle of pile foundations; and the area available for frictional resistance is reduced near the foot where the natural compression of the earth is generally the greatest and most useful. It should be added that the highly compressed and loaded area near the head of the pile may have its supporting power reduced by subsequent shallow excavations or by erosion in contiguous areas. Probably a more important objection to a large taper is that an increased bearing capacity is artificially created in the ground which becomes dissipated in time as the pressures are distributed through a larger mass. In districts subject to floods the bearing power of the ground near the surface is at least temporarily reduced, and, if a large percentage of the load is carried by the ground near the surface, serious settlement is very likely to result.

It should be remembered that in driving a straight pile the compression of the earth is done at the tip by increments as the penetration of the pile increases. On the other hand, in driving a pile with a large taper the compression thus made at the tip is materially smaller, but the compression is continuously increased all along the depth of penetration while the total resistance increases to its final maximum value. The tapered pile, however, causes less displacement or disturbance of the texture or internal arrangement of the material through which it is driven than the straight pile.

Experience in driving concrete piles into hard clay for the foundations of the Kentucky and Indiana bridge at Louisville, in 1911, led to a change in the taper by reducing the thickness of the head from 20 to 14 in., leaving the thickness of the foot the same as before, or 9 in., below which there was a pyramidal point 9 in. long. The piles were square in cross section and 22 ft. long. In some cases 5,000 blows had been required previously for the 25-ft. piles with the larger taper.

Various tests have been made to determine the effect of taper upon the resistance of a pile. In a test at Chicago a tapered steel core and an oak pile both 20 ft. long were driven within a few feet of each other. The diameters of butt and tip were 18 and 6 in. for the core, $12\frac{1}{2}$ and 10 in. for the oak pile. With a 2,200-lb. hammer falling 25 ft., the former penetrated an average of 1 in. for the last several blows, and the latter $5\frac{1}{2}$ in. The volume of the oak pile is 67.5 per cent of that of the steel core.

In incompressible but plastic clays the wedge action of tapered piles is found to be of no value according to loading tests. Extensive experience proves, however, that concrete piles with a large taper have been used successfully in compressible ground to form foundations without subsequent appreciable settlement. In many cases, doubtless, the spread footing would have been a more appropriate type of foundation. In other cases sand piles might be preferable, for, if the ground is to receive its greatest degree of compression near the surface, it would apparently be a more economical arrangement to fill the conical holes made by the tapered core with sand, since sand is less expensive than concrete and the increased bearing power of the ground could be utilized equally well by the concrete cap or footing.

The following experiment is very instructive regarding the effect of taper. A concrete pile was driven to a total penetration of 26.5 ft., the diameters at the surface of the ground and at the foot being 18.6 and 8 in., respectively. The safe load was computed to be

40.9 tons. A wooden pile was driven to a total penetration of 24 ft., the diameters at the surface and at the foot being $11\frac{7}{8}$ and $9\frac{1}{2}$ in. Its safe load was computed to be 11.6 tons. These piles were both driven in dense blue clay. They were subsequently loaded, and the test loads for a settlement of $\frac{1}{4}$ in. in each case were 44 and 32.1 tons, respectively. As the frictional surfaces are 92.4 and 67.2 sq. ft., the resistances are found to be 0.476 and 0.478 ton per sq. ft., respectively.

6-19. Static-load Tests and Pull Tests. In designing a concrete-pile foundation the same elements must be considered as when timber piles are used. The safe load may depend (*a*) on the strength of the pile acting as a column, (*b*) the frictional resistance along the sides of the pile plus point bearing, and (*c*) the bearing capacity of the soil on horizontal planes below the foot of the piling.

Simple structural analysis is sufficient to determine the necessary area of cross section to carry the load. In general it will not be necessary to treat the pile as a long column since the slenderness ratio is usually much less than in a timber column. Column action of piles is discussed in Art. 5-2. The axial compressive stress in concrete piles will seldom exceed 300 to 400 lb. per sq. in.

Where the limiting factor is frictional resistance and the soil is of a sandy nature, dynamic pile-driving formulas may be used with the same assurance as when timber piles are employed (Art. 5-5). However, static load tests give a better indication of bearing power even for sandy soils and for plastic soils they are a necessity. A very satisfactory testing outfit is shown in Fig. 5-1*a*. The concrete cap cast on top of the pile provides bearing for the 12- by 12-in. timber beams which support crossbeams and plank flooring. Wedges or jacks are placed at the outer ends to prevent the platform from tipping during loading. After the loads have been placed, the platform is balanced and the blocking lowered slightly until the entire weight rests on the pile. If sand or other loose loading material is used, the platform must be provided with sides.

Many modifications of the above noted type of loading device are possible. In testing piles for a highway trestle in California, wood piles were driven in a circle around the test pile, and a steel tank, with a capacity of 100 tons of water, was placed on the pile cluster. In testing the concrete pile the full load of the tank was transferred by jacks to the concrete pile in the center. To test concrete piles for a highway trestle across the Bonnet Carre Spillway near New Orleans, a heavy steel stirrup was hung on the head and square 5-ton concrete blocks were stacked upon it, each block having

a square hole in the center so that the lower blocks were slipped over the head of the pile..

Some interesting experiments were made on frictional resistance of concrete piles at Columbus, Ohio, in 1933. They were tested for uplift, thus eliminating the effect of point bearing. The piles were precast and were 14 in. square in section. Eight piles were pulled with the following results, the range being from 511 to 722 lb. per sq. ft.:

Piles	Penetration, feet	Frictional resistance, pounds per square foot
1	11	538
2, 3, and 4	14	557 (average)
5, 6, 7, and 8	19	613 (average)
Average of 8 piles		582

The shorter piles penetrated a sedimentary deposit which was a mixture of loam, clay, sand, and gravel. The lower ends of the long piles penetrated a bed of gravel, which accounts for the higher frictional resistance. Ground-water level was quite near the surface.

Pull tests made on Raymond conical concrete piles, averaging 16.8 ft. in length, driven through dark yellow clay, fine sand, and coarse sand at Fort Wayne, Ind., developed an average pulling resistance of 755 lb. per sq. ft. for an average deformation of 0.03 in. The load applied was 40,000 lb. A bearing load of 120,000 lb. caused a permanent settlement of 0.15 in. on these piles. The piles were driven by a single-acting steam-hammer, the weight of ram being 5,000 lb. and the stroke 3 ft. The penetration averaged $\frac{1}{4}$ in. per blow for the last 12 blows. The safe load by the *Engineering News* formula was 77,000 lb., on the basis of 90 per cent hammer effectiveness.

A 12-in. cast-in-place pile 19 ft. long, placed in loosely compacted sandy loam in Los Angeles was tested after eliminating the effect of end bearing. A 3-ft. shaft was sunk 9 ft. from the pile and a tunnel extended from it to a point underneath the pile. The lower end of the pile moved $\frac{1}{32}$ in. under a load which developed a frictional resistance of 1,170 lb. per sq. ft.

Although but few definite data are available as proof, it is probable that the frictional resistance of concrete piles is considerably greater than for wood piles driven under identical conditions. It

has also been observed that the rougher the concrete the greater the resistance.

Where there is likelihood of the safe load depending on the bearing capacity of the soil on horizontal planes below the foot of the piles, as in the case of rather soft material extending to an indefinite depth, tests should be made on a group of piles as well as on a single pile. If one obtains the relation between the group capacity and the single-pile capacity, it will be possible to approximately determine proper pile spacing or single-pile load limitation (see Art. 5-7).

Tests were made on a five-pile group of uncased 15-in. diameter pedestal piles in Washington, where the soil consisted of successive layers of (*a*) blue clay; (*b*) sand; (*c*) gravel; (*d*) a mixture of mud, clay, and sand; (*e*) gravel with clay and boulders; and (*f*) green clay and sand. The first pile was driven to a depth of 26 ft., penetrating the green clay and sand, but, as the driving progressed, the piles had to be shortened to a minimum of 17 ft., owing to the difficulties of driving due to the consolidation of the soil. The presence of the sand and gravel layers near the top prevented soil displacement by heaving, the same amounting to only about 6 in. and being confined to a space immediately around the pile tops. A total load of 250 tons on the five piles resulted in a settlement of $\frac{5}{32}$ in., and, when the load was removed, the permanent settlement was found to be $\frac{1}{16}$ in. The skin friction, including point bearing, amounted to 1,275 lb. per sq. ft.

CHAPTER VII

SAND PILES, METAL PILES, AND SHEET PILES

7-1. Sand Piles. Short timber piles are sometimes used to compact the soil and thus increase its bearing power. The same result may be accomplished at less cost by withdrawing the pile as soon as it is driven and filling the hole with sand. Such piles are called "sand piles." They can be placed without regard to the elevation of the ground-water level but cannot be used if there is any danger of scour or in regions subject to earthquakes. The use of sand columns confined in wooden boxes to lower great weights has proved that they will sustain loads while developing relatively small lateral pressures. In order to have the sand pack firmly, it should be moistened when placed in the holes and tamped. In case there is a slight settlement, the sand will readjust itself and maintain its stability.

A notable example of soil consolidation by the use of sand piles is that for the Federal Legislative Palace in the City of Mexico. Here 150,000 sand piles, 8 in. in diameter on 20-in. centers were placed in the subsoil, which consisted of an unreliable mixture of volcanic ejections and alluvial matter. The heave, or raising of the general level of the ground, amounting to only 1 ft., as compared to a possible 5 ft. had there been no soil consolidation, indicated that 80 per cent of the pile displacement was effective in consolidating the soil.

In building a warehouse in Salina, Kan., 7-ft. sand piles were placed by drilling holes through the sandy loam with a posthole auger. The sand piles were stopped 3 ft. below the ground surface and the space above filled with concrete. Load tests made on top of the sand showed $\frac{1}{4}$-in. settlement under loads of 8,000 to 11,000 lb. and 1-in. settlement under loads of 11,200 to 14,400 lb.

The "compressol system" is somewhat analogous to sand piles in that a hole is first formed and then filled in with a different material. The hole is made by a heavy conical perforator having a sharp point which is successively raised and dropped until the hole reaches a hard stratum. If the compressed earth does not keep the water out, the hole may be lined with clay dumped in

after each fall of the perforator. Boulders are dropped into the hole and rammed with a tamping rammer which is shaped like a cartridge, thus forming a layer at the enlarged bottom of the hole. Concrete is then deposited in batches and tamped in the same way. In this way a sort of rude concrete pillar is formed. The system was originated in France and is seldom used in this country. It is more economical to use concrete in the form of concrete piles as described in the previous chapter.

7-2. H-section Bearing Piles. Dating from about 1900, steel bearing piles in the form of I-beams and fabricated sections came into extensive use for the foundations of light highway bridges in the Middle West, particularly in Nebraska.[1] Here the rivers are wide but shallow and have large drainage areas, which results in heavy floods and ice floes in the spring. A severe scouring action of the floods often removes the sand and gravel of the river beds to a depth of 20 ft. or more. Steel I-beam piling was found to be a very satisfactory foundation material, for it could economically be driven to a great depth and also possessed considerable bending strength to resist the action of ice.

When the H-column section was placed on the market in 1908, it was immediately recognized as being superior as a piling material to the I-beam section. At the present time a wide range of H-sections are available, some of these being specially rolled for piling use, in which case the thickness of the web is made equal to that of the flanges. Common sizes used vary from 8 to 14 in. depth of section, inclusive, and from 33 to 117 lb. per ft. of length. It is commonly specified that (*a*) the flange projection shall not exceed twelve times the minimum thickness of either flange or web, (*b*) the flange width shall be not less than 85 per cent of the depth of the section, and (*c*) the minimum thickness of metal shall be $\frac{7}{16}$ in. The quality of material is specified to be standard structural-grade open-hearth steel, as outlined in the standard specifications of the American Society for Testing Materials.

Experience indicates that corrosion is not a serious problem, particularly when the pile does not extend above ground-water level. Sometimes it may be advisable to use copper-bearing steel with a copper content of not less than 0.2 per cent, or the section may be increased to the next weight over that required to take the load. Other precautions include shotcreting or placing a concrete covering about the piling near the ground line. The specifications

[1] Steel-pile Foundations in Nebraska, *Civil Eng.*. vol. 2. p. 553, September. 1932.

of the American Railway Engineering Association state that "piles shall be protected from corrosion by means of concrete encasement extending at least 1 ft. above ground or normal water line and 3 ft.

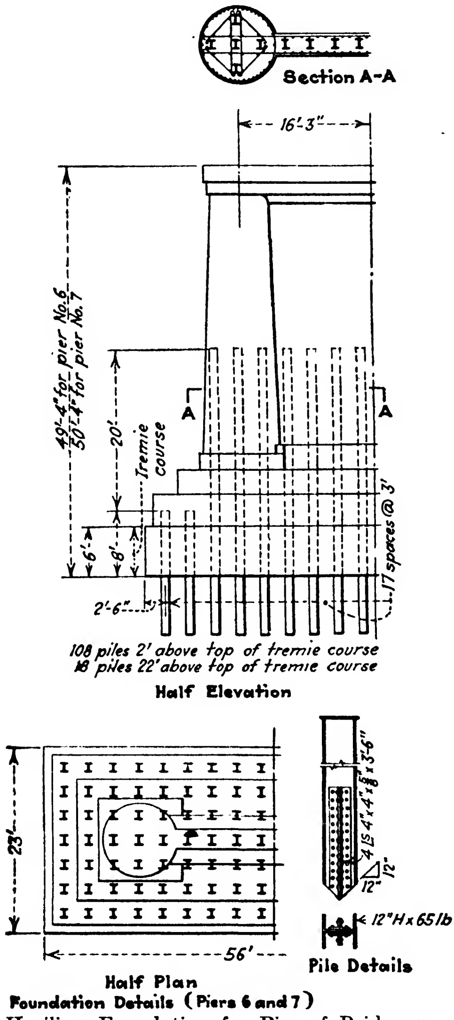


FIG. 7-3a.—Steel H-piling Foundation for Pier of Bridge over Kansas River in Kansas City.

below the ground or minimum water line. The minimum concrete cover shall be 3 in."

7-3. Types of Installations. The use of steel piling for bridge piers and abutments may be divided into the three following catagories: (*a*) piling used to support gravity structures in a manner

similar to those where timber or concrete piles are used, (*b*) piling—either encased in concrete or arranged to form pockets filled with concrete—extending nearly to the top of the pier or abutment and forming an integral part of same, and (*c*) unencased piling forming the pier or trestle bent itself and usually rigidly fastened to the superstructure by welding or riveting. For the first type of foundation, after driving the piles to the desired penetration, the tops are burned off to the proper elevation and pile caps—usually steel plates—are then welded on.

Figure 7-3*a* illustrates the first type, where the piles, instead of being cut off near the ground line, extend up some distance into the

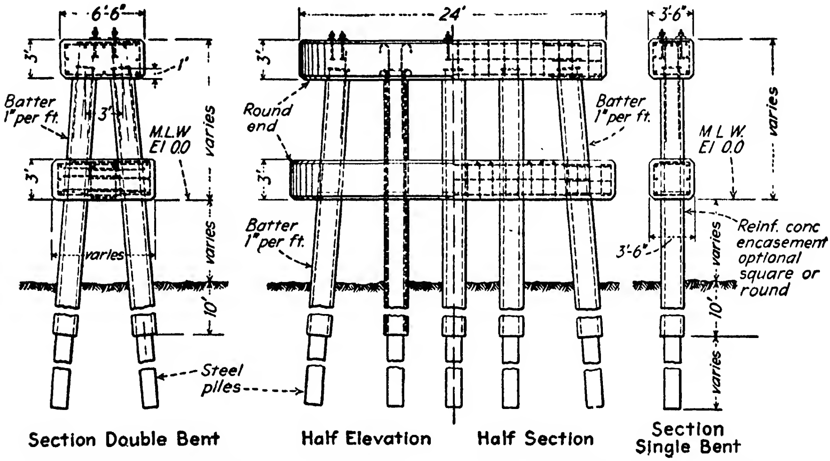


FIG. 7-3*b*.—H-piles Used in Trestle Bents of Bridge over Potomac River at Ludlow Ferry, Maryland.

pier.[1] The piles were driven to a maximum depth of 53 ft. below the footing. The design load was 35 tons per pile excluding the effect of longitudinal and lateral forces and 50 tons per pile when including the same.

The second type is well illustrated by the 59 trestle bents used on the Virginia approach of the bridge over the Potomac River at Ludlow Ferry, Maryland, in which piles of the record length of 194 ft. were used. As shown in Fig. 7-3*b*, each trestle bent consists of five steel H-section piles encased in concrete to a depth of 10 ft. below ground level.[2] Every sixth bent is a double one. Each pile is capped with a welded grid of 1- by 2-in. bars, and in each bent the piles are tied together with a concrete collar at the water

[1] See *Eng. News-Record*, vol. 111, p. 459, Oct. 19, 1933.
[2] See *Eng. News-Record*, vol. 124, p. 306, Feb. 29, 1940.

line and a concrete cap. The pile sections vary from 12 by 12 in. 53 lb. to 14 by 16 in. 102 lb. The piles were driven to a bearing value of 45 tons as determined by the Modified *Engineering News* formula (Art. 5-4), using a value of $c = 0.1$.

Figure 7-3*c* shows the type of construction for a bridge over the Tuscarawas River at Dover, Ohio, in which Z-piling was used in the shape of a double cross and filled with concrete from an elevation some distance below mud line to form piers. The piling was driven for skin-friction bearing in sand-gravel strata containing some clay.

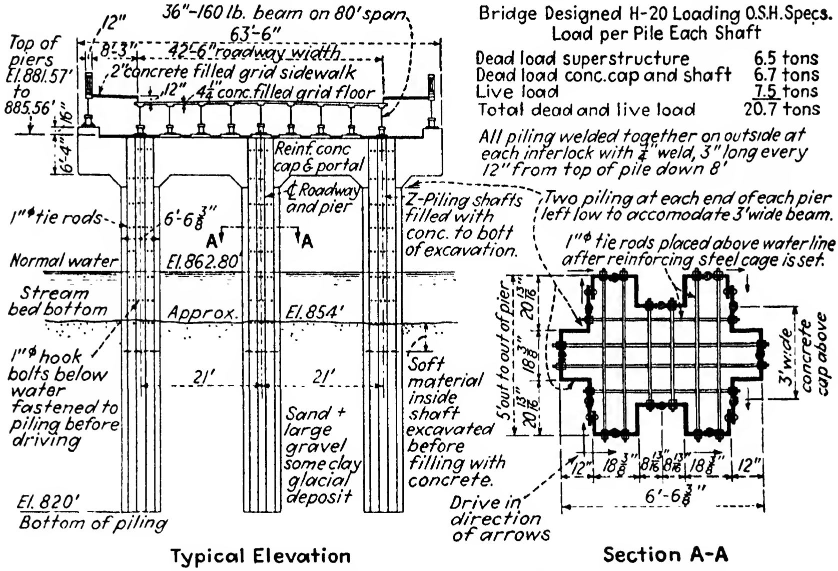


Fig. 7-3*c*.—Pier Shafts of Z-piles Filled with Concrete for a Bridge over the Tuscarawas River at Dover, Ohio.

Each pier shaft was made up of eight pieces of copper-bearing Z-sheet piling, ½ in. thick and weighing 57 lb. per lineal foot. Piles were about 65 ft. long except for the two on each side of each shaft which were 8 ft. shorter to permit continuity of the reinforcing and concrete of the pier cap.

The soft river bottom material was removed down to good gravel and sand prior to setting the piling. On the completion of driving, the sections were welded together as noted in the illustration. After cleaning out any soft material in the shaft, a prefabricated cage of reinforcement was lowered into the shaft extending from the river bottom to normal water level. Above this point 1-in. round tie rods were placed to connect opposite piling sections at 4-ft. intervals, after which the concrete filling and cap sections were placed.

Illustrating the third type are the steel piers used by the Canadian National Railways.[1] A typical pier consists of closely spaced 12-in. 54-lb. H-section piles covering the entire area of the pier and evenly spaced in rows. In the field, 15-in. 33.9-lb. channels are

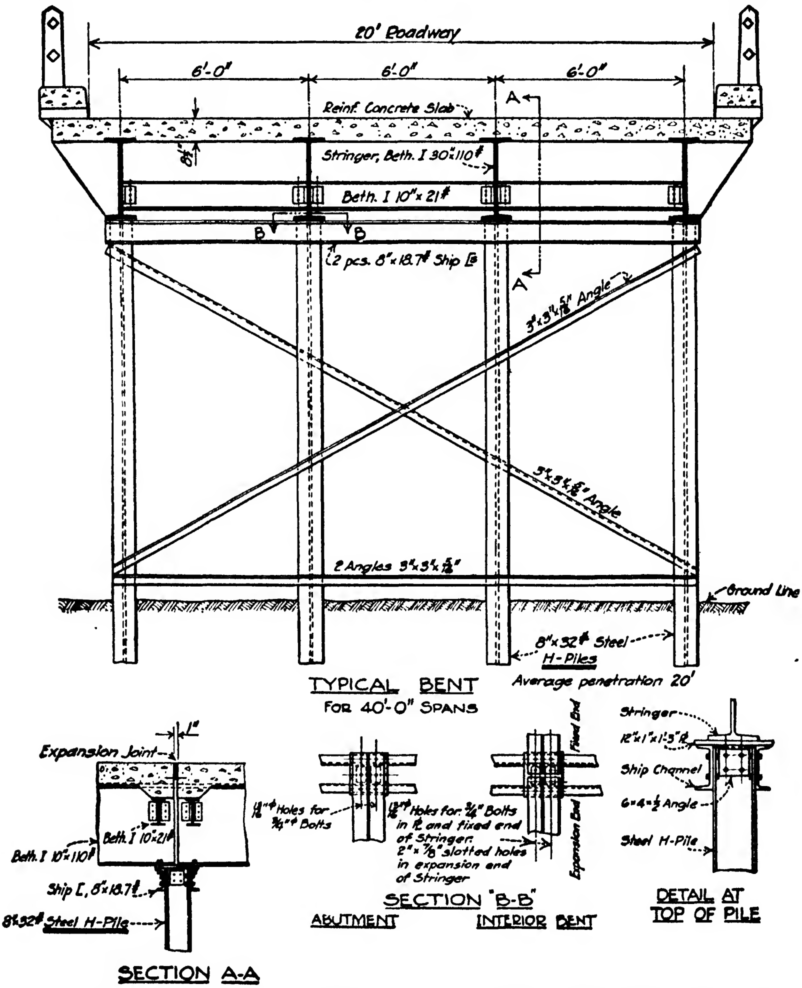


FIG. 7-3d.—Steel-pile Bent of Ponchorico Creek Bridge, Monterey County, California. (*Courtesy of Bethlehem Steel Co.*)

welded to the tops of the piles to form a cap for the bearing of the steel grillages used to support the superstructure. Bracing between water level and pile tops consists of steel angles welded to the piling and placed horizontally and diagonally to form trusses.

[1] *Eng. News-Record*, vol. 123, p. 826, Dec. 21, 1939.

Figure 7-3*d* shows the design of the Ponchorico Creek bridge in Monterey County, Calif., and also illustrates this type. The steel-bearing plates for the stringers rest on two channels fabricated at the pile tops. The diagonal bracing is welded to the outside of the flanges of the piles. In this job a considerable amount of drilling and riveting was done. However, at the present time welding details are rapidly replacing riveting for this type of work. Sometimes the piles of a bent are set up in a rigid structural frame to ensure exact fits after driving. Steel H-piling is now used not only for bridge foundations but also for building piers and wharves.

Among the advantages claimed by manufacturers for H-piling are the following: (*a*) ability to stand up under hard driving; (*b*) small soil displacement, permitting close spacing of piles; (*c*) long lengths available (now rolled in lengths up to 120 ft.); (*d*) full-strength splicing easily made by fusion welding; (*e*) high bearing value when driven to rock; (*f*) large surface area to develop skin friction; (*g*) long life; (*h*) immunity to action of marine borers; and (*i*) resistance to flotation. Piers on open sand shores sometimes fail by flotation due to the heavy pounding surf putting the sand into suspension around the piling, after which a heavy ground swell gets beneath the deck and raises the pier. Steel piling driven well below the elevation of suspended sand has been demonstrated as an excellent type of construction.

7-4. Driving H-piling. Steel H-section piling is usually driven with a steam-hammer, a heavier outfit being used than in the case of timber piles. Figure 7-4*a* shows the pile driver used by the Merritt-Chapman & Scott Corporation in placing the record-length piles of the Ludlow Ferry bridge. The A-frame, which is 90 ft. high, carries telescopic leads that can be extended 60 ft. above the top of the tower and 95 ft. below deck. In driving piles only one splice was permitted, the first section being 115 ft. long and the second section furnishing the remainder of the length to make a maximum length of 194 ft..

For most jobs the steam-hammer selected should develop not less than 7,000 or more than 15,000 ft.-lb. of energy per blow. Where the hammer is too light and the energy available insufficient, progress will be slow and much of the energy will be wasted. Where the energy is excessive and the penetration per blow small, there is danger of damage to the pile. The first sign of overdriving will be a buckling or wrinkling of the metal near the head of the pile. Damage of this nature will be reduced by using a cast-steel driving cap

(Arts. 3-11, 6-15, 7-15). The use of a cap also helps to keep the pile in proper alignment.

The bottom of the pile should not be pointed, for the square end provides a better bearing surface and does not hinder driving. In general, water-jets should not be used in sinking H-piling as the

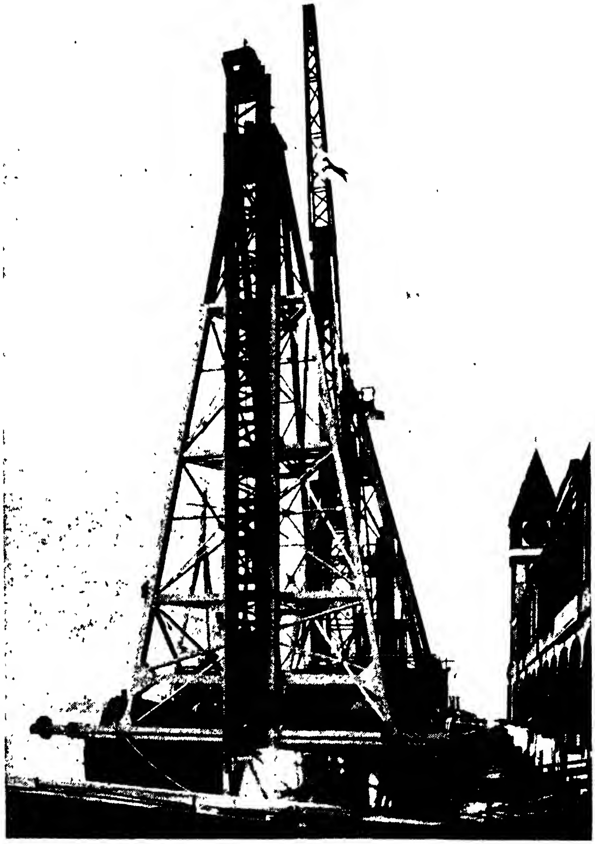


FIG. 7-4a.—Pile Driver Used in Driving Long H-piling on Ludlow Ferry Bridge, Maryland. (*Courtesy of McKiernan-Terry Corp.*)

action of the water prevents the formation of the densely compacted earth cores between the flanges. These cores, sometimes compressed to half the original volume, are very helpful in increasing the load-carrying capacity of the pile.

The cost of H-piling in place varies widely, depending on the location and difficulties involved in driving, but in general it will vary from 3.5 to 6 cts. per pound of metal.

7-5. Load Capacity of Piles Driven to Rock. The load-carrying capacity of a steel H-pile driven to rock or other hard material will largely depend on its strength as a column or on the crushing strength of the material under the foot of the pile. A widely used column formula is

$$f = 15{,}000 - \frac{1}{3}\left(\frac{l}{r}\right)^2$$

where f = allowable unit stress in pounds per square inch
l = distance in inches between points of lateral support
r = least radius of gyration of cross section in inches

In firm material a pile driven to full penetration in the ground may be properly assumed as supported laterally for its full length. However, it is customary even under this condition to limit the load to about 8,000 or 10,000 lb. per sq. in. of cross section. The total load is likewise generally limited as follows:

Size of pile, inches	8	10	12	14
Load in tons	40	45	55	65

Where soft ground overlies firm material, it is commonly specified that the load should be limited to about 60 per cent of the value found by using the above formula and taking the unsupported length as $1\frac{1}{2}$ times the length of the portion of the pile within the soft material, with the full length of the pile as the limit. When used in trestle bents, the length of the unbraced part extending above ground should be added.

A structural failure will not result when a small area of a large rock mass is loaded, even when the load is very large. However, the rock material may gradually become pulverized under repeated load applications; consequently the pressure should be limited to some reasonable figure, perhaps 6,000 lb. per sq. in. in the case of hard rocks and one-half this value for softer rocks. By the use of reinforcing plates or structural shapes welded or riveted to the foot of the pile, the unit bearing can readily be kept down to any desired value (see Fig. 7-3a).

It is generally specified that piles 40 ft. or less in length shall be unspliced section. Splices should be made with the pile sections in full contact and the splice should be of such a type that it will resist driving shock and maintain perfect alignment of the pile. A reduction of 5 per cent is often made in the allowable load on the pile for each splice in excess of one. Splices on unsupported lengths

of piles should be made to develop the full sectional strength of the piles. Where piles are driven to rock, the minimum spacing is usually specified as twice the depth of the pile section, but not less than 2 ft.

Two interesting bearing tests[1] were made at Lackawanna, N. Y., on 10-in. 57-lb. steel H-piles. They were about 26 ft. long and were driven to a penetration of 24 ft., through soft clay and 3 or 4 ft. of sand and gravel to shale. Load was applied to the test pile by a hydraulic jack acting against a steel frame loaded with steel ingots. The four steel piles supporting the frame also furnished resistance to uplift. Pronounced settlement of the first pile occurred at a load of approximately 134 tons, or about 16,000 lb. per sq. in. of cross section. At a load of 127 tons the total settlement was 0.23 in., while for 140 tons it was 0.50 in., the latter load remaining on for $1\frac{1}{2}$ hr., the total time of the test being about $2\frac{1}{2}$ hr. On removing the load there was an elastic recovery, the net final settlement being 0.34 in.

In applying the rules outlined in the first part of this article to determine the design strength of this pile, with $r = 2.45$ in. and $l = 26$ ft., we have $f = 15{,}000 - \frac{1}{3}\left(\frac{26 \times 12}{2.45}\right)^2 = 9{,}590$ lb. per sq. in. For a sectional area of 16.76 sq. in., the design load is $16.76 \times 9{,}590 \times 0.6 = 96{,}400$ lb., or 48.2 tons. As this is larger than the specified maximum of 45 tons, the latter will govern, and so the factor of safety is $\frac{134}{45} = 3$.

7-6. Load Capacity of Friction Piles. In those cases in which no hard stratum is reached, the load will be carried by a combination of side friction and point bearing. The percentage of the load carried by side friction will be much greater in the case of H-beam steel piles than for wood or concrete piles, due to the large difference in the ratio of the perimeter to the sectional area. For example, for a 15-in. cylindrical concrete pile this ratio is 0.27, whereas for a 10-in. 42-lb. H-beam it is approximately 4.75 (based on the gross perimeter), or 3.21 (based on the net perimeter, that is, the perimeter of the circumscribing rectangle). The latter is the more logical figure to use where earth cores form between the flanges of the H-pile.

Pile-driving formulas may be used to determine approximately the load-carrying capacity of steel piles when driven in soils of a sandy nature, but, as explained in previous articles, no dynamic

[1] See bulletin entitled "Bethlehem Steel H-Piling" (catalogue 133-A), p. 38, Bethlehem Steel Company.

formula is of much value in plastic soils, owing to the effect of the time element. In driving steel piles, the ratio of the weight of the hammer to the weight of the pile is generally much less than when driving timber piles; hence any formula used (Art. 5-4) should take this fact into consideration.

Where pile-driving formulas are not satisfactory, static test piles should be used. It is generally specified that a load $1\frac{1}{2}$ times the design load, standing 24 hr., shall show no continuous settlement and that the total net settlement, after deducting rebound, shall not exceed 0.01 in. per ton of total test load. Design loads for friction piles are usually limited to 30 tons. The exceptions are where the penetration exceeds 40 ft. or where the pile is driven to refusal in hardpan or in gravel or boulder formations not underlaid by a softer stratum, in either of which cases the load per pile may be 40 tons.

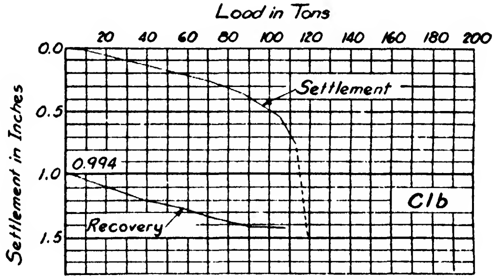


FIG. 7-6*a*.—H-pile Test on San Francisco-Oakland Bridge Falsework.

In 1935 the Bethlehem Steel Company made a series of load tests on some 12-in. 65-lb. H-piles which had been used as a part of the falsework footing in erecting the San Francisco-Oakland bridge. These piles had been in use for 3 or 4 weeks before the tests were made, supporting loads of about 20 tons. The loads were released about 2 to 3 weeks before testing. The depth of water was 40 ft., and the material penetrated consisted of 13 ft. of surface mud, 20 ft. of clayey sand, and then sandy, silty clay. The driving was done with a double-acting hammer furnishing 19,000 ft.-lb. of energy per blow and having a weight of ram of 5,000 lb.

Figure 7-6*a* shows the load-settlement diagram[1] of one of the piles which was 98.8 ft. long and had a penetration of 40.5 ft. The yield load was considered as that load at which the slope of the load settlement was $4\frac{1}{2}$ times the slope of the elastic deformation line of the steel alone. The elastic deformation was based on an assumed length of 80 ft., or one-half the penetration plus the length

[1] See *Booklet* 99, p. 44, Bethlehem Steel Company.

of pile above ground. For this pile the yield load was taken as 104 tons. For this load the elastic deformation would be 208,000 × 80 × 12/(19.11 × 30,000,000) = 0.35 in. It will be noted from the diagram that the recovery was only slightly more than this figure. The penetration under the last blow of the hammer in driving was 0.141 in.

If the Boston Code formula (Art. 5-4) is applied to this pile, assuming R_a = 76,000 lb., then

$$K = \frac{1.5 \times 76{,}000 \times 98.8 \times 12}{19.11 \times 30{,}000{,}000} - 0.05 = 0.286 \text{ in.}$$

$$r = \frac{5{,}000}{6{,}500} = 0.77$$

and

$$R_a = \frac{4 \times 19{,}000}{0.141 + 0.286}\,\frac{0.77}{1.77} = 77{,}500 \text{ lb.}$$

By the Modified *Engineering News* formula (Art. 5-4),

$$R_a = \frac{2 \times 19{,}000}{0.141 + \dfrac{0.3}{0.77}} = 71{,}500 \text{ lb.}$$

The force required to pull this pile was 164,000 lb., or 1,000 lb. per sq. ft. of frictional area, basing the frictional area on the perimeter of the circumscribing rectangle.

7-7. Pile Attachments. Various types of attachments are used to increase the load-carrying capacity of H-piling. The simplest type of attachment, called "core stoppers," consists of horizontal plates with suitable stiffeners welded between the pile flanges and spaced at suitable intervals along the pile. These core stoppers serve to consolidate the earth cores and keep them in place. The load tests mentioned in the preceding paragraph included some made on piles with core stoppers, varying from one to three pairs. The results did not indicate any advantage in the use of these, for highest bearing values were obtained for plain piles.

Other types of attachments used to increase the frictional area and to compress the surrounding earth consist of timber or metal lagging in lengths of perhaps 10 ft. Timber lagging consists of two sticks of timber fitted between the flanges of the pile unit and bolted through the web. Steel lagging usually consists of short lengths of the piling section welded to the main pile, flange abutting against flange.

7-8. Tubular Piles. Metal pipes, filled with plain or reinforced concrete, have been used to a limited extent for bridge foundations and quite widely for building foundations, especially where underpinning is involved (Chap. XVII). The steel pipe or casing can be of any diameter of which pipe and well casings are manufactured, 10- to 18-in. diameters being common sizes and used under trade names such as Hercules piles and Tuba steel cylinders. Diameters as large as 54 in. have been used, but, where the pipe is large enough to admit workmen to excavate the interior by hand, they are generally regarded as caissons (see Arts. 9-5, 9-6, 10-8, 11-3). The thickness of pipe is usually from $\frac{1}{4}$ to $\frac{1}{2}$ in., although a thickness of $\frac{7}{8}$ in. for 12-in. pipe has been used in driving through sunken timber cribs and boulders. The American Railway Engineering Association specifies that "pipe shall have an inside diameter of 10 in. or more and a thickness of not less than $\frac{3}{8}$ in., except that a thickness of $\frac{5}{16}$ in. may be used on 10-in. and 12-in. pipe."

Pipe is used in lengths of about 20 ft., one or more sections being used as required. Cast-steel inside sleeves (Fig. 7-9*a*) having a length at least twice the diameter of the pipe are commonly provided to hold the sections together, and they have a driving fit in the piles. The ends of the pipe sections are machined so as to be truly perpendicular to the axis, thus securing a true alignment of the pile and a uniform bearing of the metal. Sometimes the sections are joined together by threaded sleeves. The tube is built up as it is driven down, and, if any material length projects above the ground, it is cut off and used on another pile.

The two general methods of sinking the casing are (*a*) driving with a steam-hammer or pneumatic-hammer (Fig. 7-8*a*) and (*b*) jacking. Jacking is generally restricted to the placing of tubes under existing buildings, although in recent years steel-tube foundations have been placed in new work simultaneously with steel erection. Casings may be driven with open ends or they may be provided with conical steel or cast-iron points. In the first case the material is generally removed by compressed-air blowpipes or by water-jets. Before the pile is filled with concrete, an electric light can be lowered to ascertain if the true alignment of the casing has been maintained.

If the casing is driven into soft soil without a shoe, the concrete filling may be rammed out at the bottom to form a bulbous foot, which increases the bearing area. In one instance, borings showed that a bed of quicksand 25 ft. deep overlaid a stratum of very coarse gravel charged with water under a high head. After an 8-in. tube

was driven down until it rested on the gravel and was cleaned out with a jet, a 1-in. pipe perforated at the bottom for 2 ft. was driven 3 ft. into the gravel; grout was forced through the pipe to form a solid footing of grouted gravel and to seal the tube, which was then pumped out and filled with concrete.

FIG. 7-8a.—Driving Tubular Piles with Steam-hammer. (*Courtesy of McKiernan-Terry Corp.*)

When the concrete is to be reinforced, special care should be taken to place and hold the reinforcement in its correct position. In filling the casing with concrete, the general practice is to place a few feet at the bottom through water by means of a special bucket. After this hardens, the water is pumped out of the tube and the remainder of the concrete, usually a 1:2:4 mixture with a maximum size of aggregate of about $\frac{3}{4}$ in., placed in the dry. The sealing concrete usually carries about 25 per cent excess cement.

This type of foundation is usually used where bedrock or hardpan is from 20 to 50 ft. below the cellar floor, although depths of 100 ft. and more are quite common. The record depth is 250 ft. below water level as noted in the next article.

For tubular piles carried by side friction, the safe loads are determined as for timber piles. For piles driven open ended to rock, the New York City Code allows loads varying from 55 tons for 10-in. inside diameters to 150 tons for 22-in. outside diameters. These values are based on a shell thickness of $\frac{3}{8}$ in. and are modified slightly for other thicknesses.

The specifications of the American Railway Engineering Association limit the stress in the concrete to 400 lb. per sq. in. and in the steel shell to 6,000 lb. per sq. in., the outer $\frac{1}{8}$ in. of metal being deducted from the thickness. Where the ratio of the pile length to diameter exceeds 40, for each diameter in length in excess of 40, the total load capacity as determined by the above rule must be reduced by 1 per cent. If splices below the upper section are closer than 20 ft., the allowable load on the pile is reduced 5 per cent for each splice in excess of the number required for 20 ft. spacing.

Experience indicates that, if the earth surrounding the piles remains undisturbed, the casing will last indefinitely. These pile casings are not good, however, when exposed to the action of moving water and air. In designing piles, careful consideration should be given also to the probability of injury due to electrolysis and to methods of protection against it. An inspection of some solid-section circular steel piling[1] after 31 years' service in the Gulf of Mexico showed a loss of metal varying from $\frac{1}{2}$ in. at the top to $\frac{1}{16}$ in. 4 ft. from the top, mean low-water level being approximately 3 ft. from the top, with a rise and fall of tide of about $1\frac{1}{2}$ ft. Wrought-iron piles,[2] 10 in. in diameter and $\frac{1}{2}$ in. thick, used for supporting a dock in San Francisco Bay showed a loss of less than $\frac{1}{8}$ in. after 50 years of service. There was no marked difference between the corrosion below low water, between high and low water, or above.

7-9. Examples of Tubular Piles. Figure 7-9*a* shows a wall pier of a 12-story office and loft building built in 1912–1913 in New York City in which three tubular piles support a wall column seated on an I-beam grillage. The inside diameter of the steel tubes is 12 in., and they are spaced 2 ft. between centers. Additional piles are driven between the clusters to carry the walls between columns.

[1] *Eng. News-Record*, vol. 111, p. 593, Nov. 16, 1933.

[2] *Eng. News-Record*, vol. 108, p. 397, Mar. 17, 1932.

In placing foundations in 1931 for the Starrett-Lehigh Building[1] in New York City, where the column loads were in some cases in excess of 2,000 tons, steel cylinders from 10 to 24 in. in diameter and $\frac{3}{8}$ to $\frac{1}{2}$ in. thick were driven with open ends from 60 to 140 ft. to rock, excavated, cut off to the required grade, filled with concrete, and capped with reinforced concrete as shown in Fig. 7-9*b*. Where a number of sections of tubing were used, the internal sleeves were

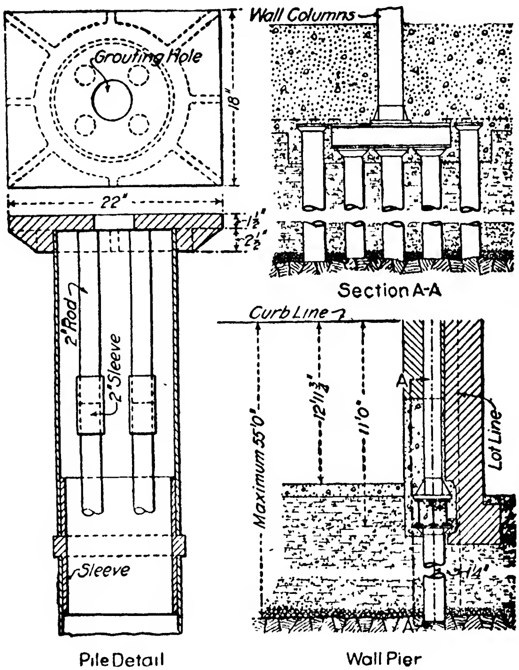


Fig. 7-9*a*. —Tubular Piles.

about three times the length of sleeves ordinarily used and had outside diameters greater than the inside diameters of the piles, with tapers at top and bottom, so that the pipe was stretched when it was driven over the sleeves. In general, heavy single-acting steam-hammers proved more satisfactory for driving the long, heavy piles than double-acting hammers. Once started, the piles were driven home to rock without stopping except for the few minutes required to add a sleeve and another length of pipe. Any delay greatly increased the difficulty of driving, particularly with the double-acting hammers.

[1] *Eng. News-Record*, vol. 109, p. 5, July 7, 1932.

In blowing out the piles a blowpipe with a gooseneck and hose leading to a reservoir of compressed air was dropped down into each pile, penetrating the material a few feet, after which by the use of a quick-opening valve the air at 100 lb. pressure threw out a mass of the soil. Water was run into the pipe in sufficient quantity to soften the mass and form a seal, so that the air would lift a reasonable volume and not merely blow out a small hole. Usually the surface of the rock was soft enough to permit the piles to be driven into it somewhat, thus forming a seal against the inflow of material when the piles were blown out, the latter usually being done in two stages, first when the piles had penetrated about 60 ft. and second at full depth. As soon as the piles were blown out to the bottom, they were inspected, sealed with a plug of concrete and later concreted.

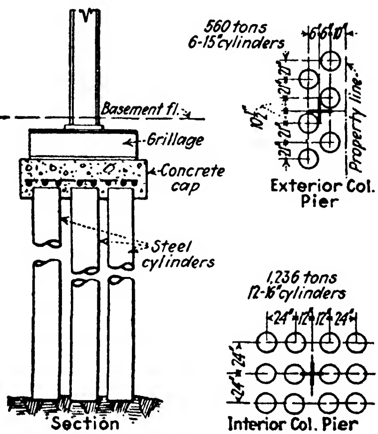


FIG. 7-9b.—Steel-pipe Pile Piers for Starrett-Lehigh Building in New York City.

When the building was completed, a few piles were tested with hydraulic jacks with a load of 192 tons, 50 per cent in excess of the design load, with a resulting settlement of only $\frac{1}{8}$ in.

In placing foundations for three bridges over the Cuyahoga River in Cleveland, 30-in. cylinders were driven through soft ground to a depth of 150 ft. below river level. The cylinders consisted of spirally welded steel pipe with shell thicknesses of $\frac{1}{2}$ to $\frac{5}{8}$ in. Each cylinder came to the job in two sections, the lower section being fitted with a tool-steel shoe having a tempered cutting edge and the abutting ends fitted with collars to hold the two sections in alignment when welded. The soil inside the cylinders was removed by driving a coring tool consisting of a 22-in. pipe into the soil, lifting it out, and ejecting the contained plug with steam.

On reaching rock a 30-in. hole was churn-drilled into the rock to a maximum depth of 9 ft. A bar reinforcement cage consisting of twenty-four $1\frac{1}{4}$-in. square bars was placed in the rock socket and for a height of 3 to 5 ft. up into the pipe to replace in steel area the cross section of the pipe. Before placing the 5,000-lb. concrete filling, the cylinders were inspected by lowering a man into the same. The maximum permissible load was 650 tons. In some

cases steel H-beams up to 120 lb. per foot in weight were placed in the cylinders to increase their load-carrying capacity.

The deepest tubular piles ever placed were those for the ventilating shaft[1] of the Holland Tunnel under the Hudson River, where the cylinders were 250 ft. long, 24 in. in diameter, and $\frac{3}{8}$ in. thick, placed through 30 ft. of water and 220 ft. of soil to rock, and then cut off by an internal cutting machine 100 ft. below water surface.

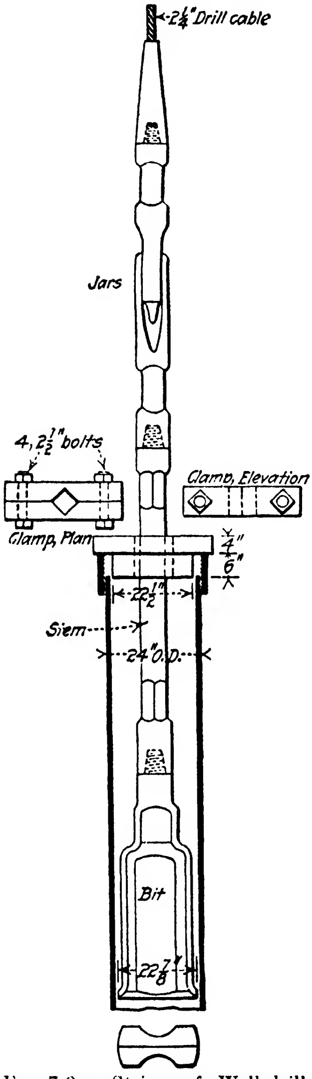


FIG. 7-9c.—String of Well-drilling Tools.

Sinking operations were conducted from a platform 6 ft. above mean high water, three sections of 20-ft. lengths first being assembled with collars on the outside and threaded connections. These three lengths sunk of their own weight to the level of the platform. A well-drilling machine of the churn type then dug 20 ft. below the bottom of the lowest section of pipe, after which another section was added and the whole casing then driven 20 ft. As explained in Art. 1-10, in this type of machine the drilling is done by a heavy cutting bit at the end of a set of tools (Fig. 7-9*c*), the up-and-down motion being effected by a machine at the surface operated on the walking-beam principle. The soil is churned into suspension by the action of this machine and from time to time this soil is raised to the surface by a bailer (Fig. 7-9*d*). The casing is usually driven by bolting to the string of tools a heavy steel clamp which strikes on a cap on the top of the pile, but for this particular job a specially designed hammer was used.

After sinking the tubes to rock, they were cleaned out by means of a water-jet through a 3-in. pipe fitted at its lower end with an

[1] *Eng. News-Record*, vol. 90, p. 242, Feb. 8, 1923.

inverted funnel, which for the 24-in. tubes had a diameter of 19 in. By this device the rising water was given a high velocity. A plug of rich concrete was then deposited by a special bottom-dump bucket. Later, reinforcement was assembled and placed in units about 20 ft. long, the longitudinal reinforcement consisting of six bars, three of which were 1¼-in. square bars and three 1⅛-in. square bars. The spiral hooping consisted of ⅜-in. round rods with a 9-in. pitch. The main reinforcement was lapped 3 ft. 6 in. and held by two U-bolts at each splice. The concrete filling was a 1:2:4 mixture and extended up to the elevation of cutoff.

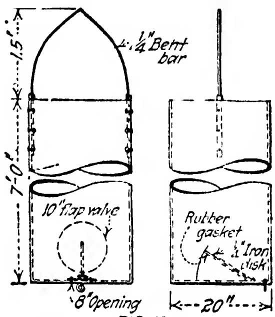
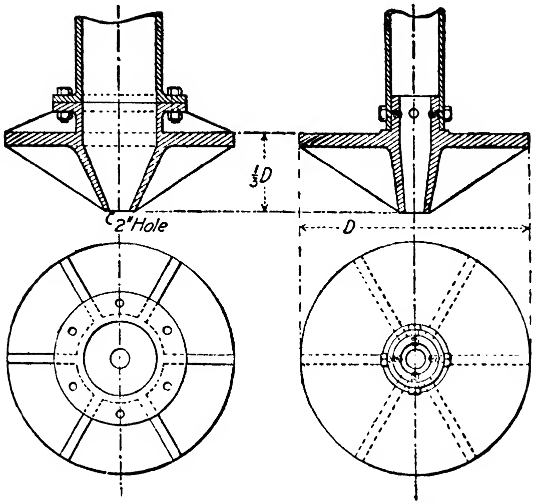


FIG. 7-9*d*.—Bailer Used to Unwater Pipes.

The design dead load was 97 tons per pile, which value was increased to 126 tons under wind action. The stresses in the concrete resulting from these two loads were 340 and 440 lb. per sq. in., respectively, on the basis of the stress per square inch in the reinforcement being fifteen times that in the concrete and taking no account of the steel in the shell except to include its cross section as an equal area of concrete.

7-10. Disk and Screw Piles. A disk pile is one which has a disk attached to its foot to provide a larger bearing area. Disk piles have been used principally in ocean piers and wharves, where the total penetration is not large and is subject to more or less variation. The minimum penetration should not be less than about 6 ft. below any possible scour. The disk is a casting which consists of a horizontal circular plate, stiffened by a number of radial ribs and connected to a central hollow stem, as shown in Figs. 7-10*a* and *b*. The former illustrates the connection of the disk to a flanged cast-iron pipe which forms the body of the pile, and the latter the connection to a steel pipe. The upper part of the stem is cylindrical, while the lower part is conical so as to form the nozzle of a water-jet or to permit a water-jet pipe to pass through it. Sometimes the ribs on the upper side of the disk are made higher than the lower ones, their edges being inclined at an angle of 45 deg. The disk pile can be used only in sand or soft material which permits sinking by the water-jet. If some material is encountered which is not easily displaced by the jet alone the pile may be rotated to cause the ribs to act as cutters. The diameters of disks range from 1.75 to 4 ft.; those of the cast-iron pipe from 8 to 14 in., with a thickness of ⅝ to 1 in.; and of the steel pipe from 6 to 10 in., the



FIGS. 7-10*a* and *b*.—Two Forms of Foot of Disk Pile.

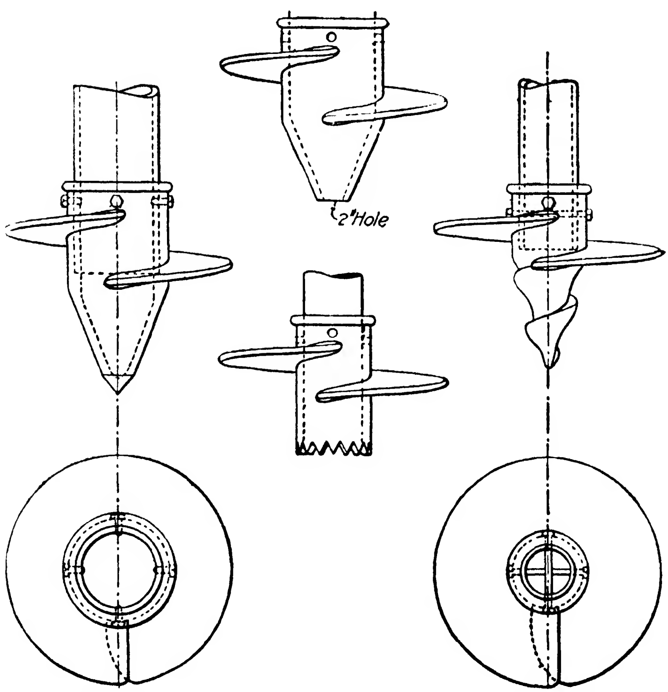


FIG. 7-10*c*. FIGS. 7-10*d* and *e*. FIG. 7-10*f*.
Four Forms of the Foot of a Screw Pile.

thickness being $\frac{1}{2}$ in. in all cases. The thickness of the disk plate, ribs, and thickest part of the stem is not to be above $1\frac{1}{4}$ inches for a diameter of 2 ft. or less, and $1\frac{1}{2}$ in. for larger diameters of disk. The ends of the cast-iron pipe sections are to be machined so as to secure perfect alignment.

A screw pile is one which has a broad-bladed screw attached to its foot to provide a larger bearing area. The use of the screw pile is similar to that of the disk pile. The form of the screw casting is illustrated in Figs. 7-10*c* to *f*. The pitch of the screw varies from one-third to one-sixth of its diameter, the pitch adopted in any case depending upon the difficulty of securing penetration. The points of the screws are also varied, the gimlet point being suitable for gravel, the blunt point for sand, the hollow conical point for the use of a water-jet in sand and gravel, and the serrated point for soft rock or coral. The dimensions of the shaft of the pile, and of the screw and its connections, must be carefully designed to furnish the torsional strength required to sink the pile into position. In one case where the frictional resistance was so great as to break several piles by torsion, it was discovered that by discharging a water-jet on the upper surface of the screw blade the friction was reduced so that the sinking could be accomplished without difficulty. After using the jet, only about one-tenth as much power was needed to rotate the piles.

Screw piles were first used in 1838 and disk piles in 1856. They are unsuitable for deep foundations where the overlying material is soft or liable to scour, since it is impossible to brace the piles below the surface. Both of these types are now of interest chiefly from a historical standpoint.

7-11. Timber Sheet Piling. Sheet piling consists of special shapes of piles driven in close contact to form a reasonably tight wall, in order to prevent the leakage of water and soft materials, or to resist the lateral pressure of the adjacent ground. Sheet piles are made of timber, of steel, and of reinforced concrete. Sheet piling is to be distinguished from "sheeting" which is set in place or driven as the excavation proceeds, as in trenches or open wells. Sheet piling is driven in advance of and usually beyond the final depth of the excavation.

The best form of timber sheet piling is known as "Wakefield sheet piling" and has been very extensively employed in this country. The patents secured in 1887 and 1891 have expired. It consists of three planks fastened together so as to form a tongue on one edge and a groove on the other (see Fig. 7-11*a*). The planks

are connected by two bolts at intervals of about 6 ft., but spikes are used at intermediate points about 18 in. apart. It has been customary to use $\frac{1}{2}$-in. bolts for planks from $1\frac{1}{2}$ to $2\frac{1}{2}$ in. thick, and $\frac{5}{8}$-in. bolts for planks 3 to 4 in. thick. For sheet piles made of 1-in. boards, $\frac{3}{8}$-in. bolts may be used. For the thin boards or planks the tongue is made $1\frac{1}{2}$ in. longer than its thickness, whereas for the thickest planks the length is the same as the thickness. The usual width of the planks is 12 in., except for those less than 2 in. thick. By sizing the middle planks to a uniform thickness, a good fit can be secured between the tongues and grooves.

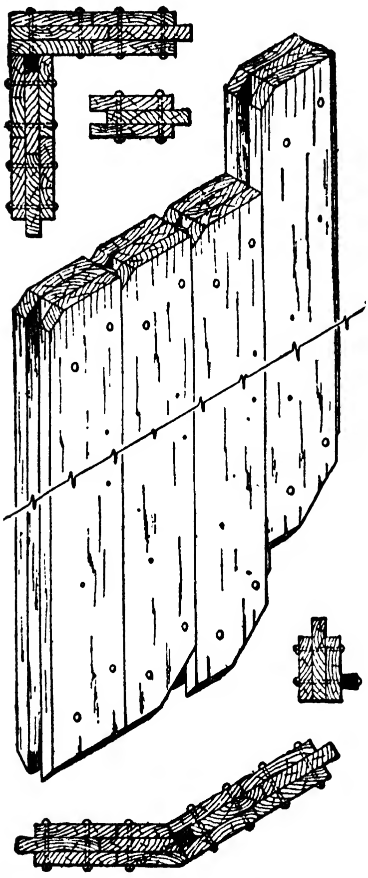
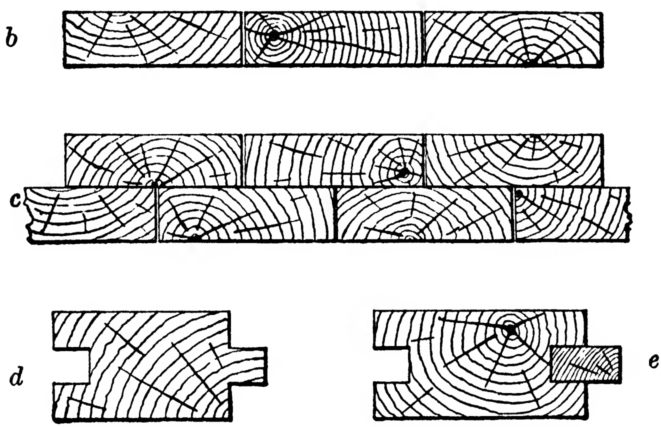


FIG. 7-11*a*.—Wakefield or Triple-lap Timber Sheet Piling.

Experience has shown that these triple-lap piles are stronger to resist driving than if made of a single stick, this being due in part to the fact that cross grain, knots, or other defects in the three planks are not likely to be located at the same part of the length; and in part that some defects become visible and lead to the rejection of a plank which might not be visible in a single stick of the same total thickness. Other advantages of this form of pile are the absence of waste in forming the tongue and groove and less tendency to warp or bend before they are driven. Figure 7-11*a* also shows how corners may be turned at a right angle by bolting and spiking a tongue to the face of a pile or at any other angle by fastening a tongue to a beveled side. It also illustrates how the foot of each pile is beveled on both faces, in order to drive plumb, and on one edge so as to keep in close contact with the adjacent one. The tongue should always be kept in the lead, otherwise gravel or stone may become wedged in the groove and damage the succeeding pile. If it is desired to drive the sheet piling each way from the

corner, the first pile should be constructed with a tongue on both sides, and sharpened so as to drive plumb longitudinally as well as laterally with respect to the lines of piling. The last pile in the center is constructed of the proper width and acts as a wedge to tighten the line if necessary. When the sheet piling is to be driven to rock bottom, the middle plank should be cut off square at the



Figs. 7-11*b–e*.—Sections of Timber Sheet Piling.

end, so that water will not readily pass underneath the piling along the rock surface.

Some other sections of sheet piling are shown in Figs. 7-11*b*, *c*, *a* and *e*, 7-11*f*, and 7-11*g*. In Fig. 7-11*b* a single row of ordinary planks is driven edge to edge. This arrangement cannot be used if it is necessary to secure a watertight wall. In Fig.7-11*c* two rows of planks are placed in contact and breaking joints. Figure 7-11*d* shows the cross section of a sheet pile which is merely a plank with an ordinary planed tongue and groove. In Fig. 7-11*e* the plank has a groove cut on both edges and a tongue formed by nailing a strip or spline into one groove to form a tongue. In this case, the tongue can be made of a different species of tough wood, like maple or elm, and carefully selected.

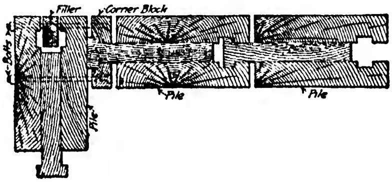


Fig. 7-11*f*.—Interlocking Wood Sheet Piling.

Figure 7-11*f* illustrates what is probably the tightest form of sheet piling. This type has been extensively used in levee work in Louisiana.

Figure 7-11*g* gives the details of construction for timber sheet piling 4 in. thick according to the standard adopted by the Southern Pacific Company. The strips nailed to the planks are beveled to form a dovetailed tongue-and-groove joint. Sheet piling built up in a similar manner is sometimes made as thick as 12 in. and in exceptional cases 15 in.

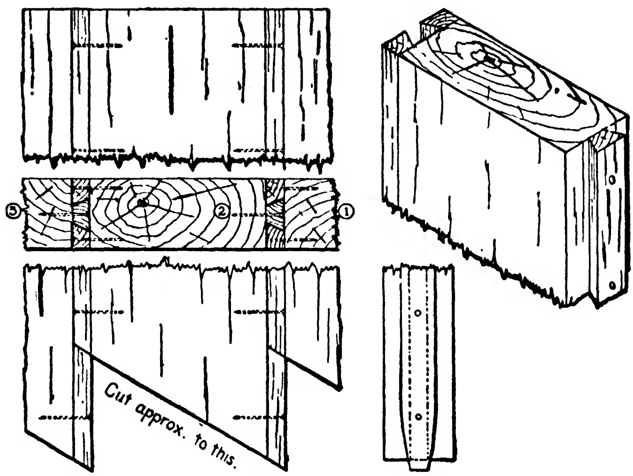


FIG. 7-11*g*.—Details of Timber Sheet Piling with Dovetail Joints.

7-12. Early Forms of Steel Sheet Piling. Although not widely used until the beginning of the twentieth century, metal sheet piling dates back to 1822 when a patent was taken out in England for a cast-iron sheet pile. Later the introduction of steel on a commercial basis gave renewed impetus to the use of metallic sheet piles. However, the early forms suffered from two defects, (*a*) unsatisfactory or very costly connections and (*b*) low bending strength for the weight of metal used.

It was not until the beginning of the present century that satisfactory types were evolved. These types may be divided into two classes as follows: (*a*) that in which standard structural-steel shapes, such as I-beams, channels, Z-bars, and plates, were fabricated to form the piling; and (*b*) that in which specially rolled shapes, particularly with respect to the interlock, were developed. The use of fabricated sections has now almost disappeared, special rolled sections being universally employed.

The first application in this country of fabricated piling was for a bridge in Chicago in 1901, the section used being similar to that shown in Fig. 7-12*a*. Alternate piles consisted of I-beams, and the

others were built up of pairs of standard channels bolted together with pipe separators, the design being based on a foreign patent.

The next step forward was taken during the following year by the introduction of the Friestedt interlocking channel-bar piling. Alternate piles were standard channels, while the other piles were built up by riveting two Z-bars to a channel. An improved form, known as the "symmetrical interlocking channel-bar piling," was later developed. In this form a continuous Z-bar was riveted along one flange of the channel, together with a short Z-bar clip along the other flange at the upper end only. This arrangement made every pile alike and preserved the symmetrical head to receive the blows of the hammer (see Fig. 7-12*b*).

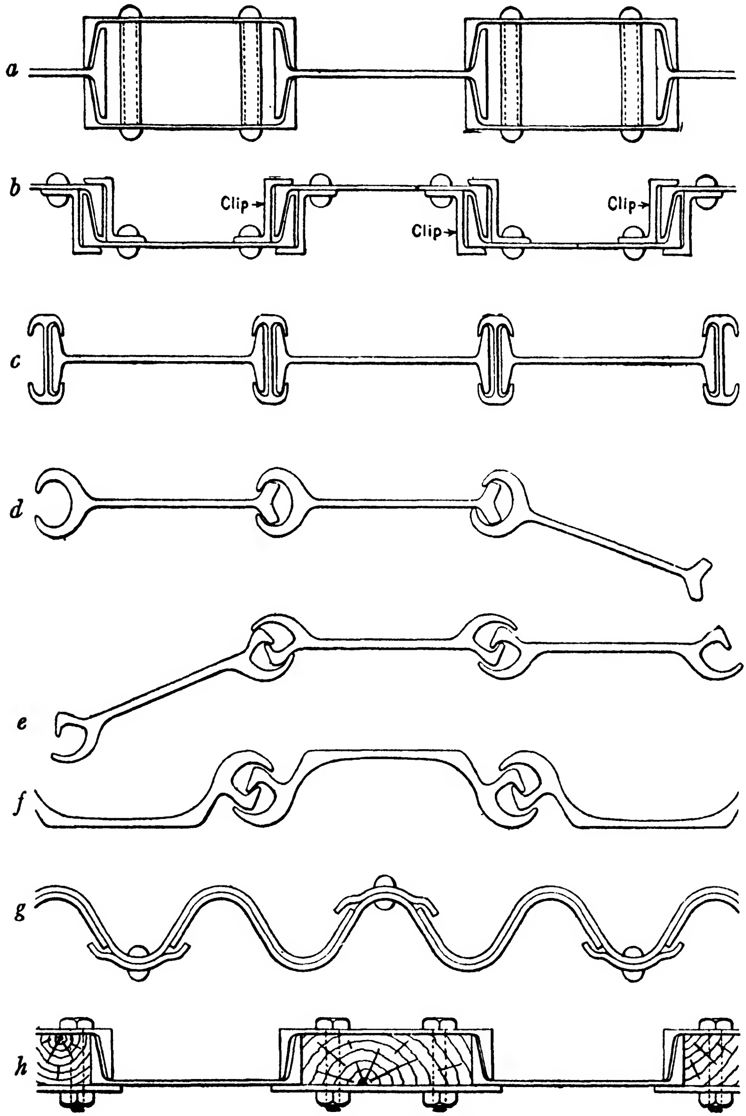
The experimental work done by Luther P. Friestedt in 1899, which led to his patented form, and his efforts to extend its use by others fairly entitle him to be known as the pioneer of the steel sheet-pile industry in this country.

Figure 7-12*c* shows a type placed on the market in 1908, which consisted of I-beams locked together by what were known as "locking bars." These bars consisted of small I-beams which, by extra passes through the mill, had their flanges bent into hook shapes. One locking bar was attached to each beam at the mill by steel wedges and then the beam and locking bar were driven as one unit. The beams differed slightly from the ordinary standard in having the outer corners of the flanges rounded. This form of piling is no longer made in this country but as manufactured in England is known as the "universal joist steel sheet piling."

Corrugated-steel sheet piling, which was first used in 1907, permits the use of thin plates but its application is generally limited to light work, such as trench bracing for sewer and pipe-line construction. Figure 7-12*g* illustrates this type as used in the early years. At present only a single thickness of metal is employed, and for temporary use locking clips are welded along one edge, whereas for permanent installations a rolled interlock is provided. The maximum thickness of metal used is $\frac{1}{4}$ in.

Figures 7-12*d*, 7-12*e*, and 7-12*f* illustrate the earlier types of piling in which the interlock was rolled into the shape. The first type, known as the "United States steel sheet piling," had one flange consisting of a bulbous section, while the other flange consisted of an open or slotted cylindrical section. This type, which is no longer rolled, was invented some years before the Friestedt type but did not come into commercial use until some years later.

The Lackawanna steel sheet piling, introduced in 1908, was also a special rolled section (Fig. 7-12*e*). Both flanges were alike,



FIGS. 7-12*a–h*.—Early Forms of Steel Sheet Piling.

the section being symmetrical with respect to a central transverse plane. The diagram shows how the flanges engaged themselves to form a double interlock. Where flexural strength was of primary

importance, the web, instead of being straight, was curved or arched as indicated in Fig. 7-12*f*.

7-13. Newer Forms of Steel Sheet Piling. Most of the steel sheet piling used at the present time (1941) in this country resembles the Lackawanna type, described above, but with minor differences in shape of interlock and in general dimensions. Figure 7-13*a* illustrates the nine sections rolled by the Carnegie-Illinois Steel Corporation. The widths—driving distances per pile—vary from 14 to $19\frac{5}{8}$ in., the weights from 30.7 to 43.8 lb. per lin. ft., and the section moduli from 3.2 to 20.4 in.^{3} per pile.

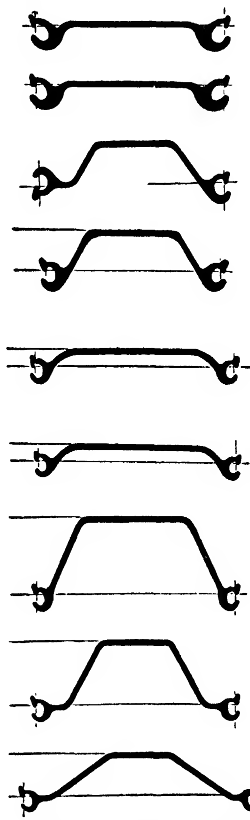


Fig. 7-13*a*.—Newer Forms of Steel Sheet Piling.

The flexibility of the interlock permits the piling to be driven in a curved line, a swing of as much as 10 deg. being possible at each interlock. This feature, together with the high tensile strength of the interlock, makes the straight web type particularly adaptable for circular cofferdam construction (Art. 8-11), where the principal stress is circumferential tension. The minimum interlock strengths vary in the different sections from 8,000 to 12,000 lb. per lin. in.

The arched types are used where stiff walls are needed to resist lateral fluid and earth pressures as in the case of bulkhead or straight-wall installations. The general method of forming a corner pile is to bend the web to a curve of short radius or to cut an ordinary sheet pile longitudinally into two halves and then connect the same by riveting to a structural angle. At the junction between a longitudinal wall and a transverse wall on one side of it, a half section like that used at a corner is riveted to the web of a whole section by means of two angles.

Where piling is needed which has a higher section modulus than is obtainable in the regular shapes, Z-piling, as illustrated in Fig. 7-13*b*, may be used. One section is rolled which has a width of 18 in., a weight of 57 lb. per ft., and a section modulus of 70.2 in.^{3} per pile.

Figure 7-13*c* illustrates the type known as the "Larssen steel sheet piling," which is a foreign design.

Further details regarding the properties and uses of steel sheet piling, proposed specifications, and methods of design may be found in manufacturers' catalogues. special attention being called to

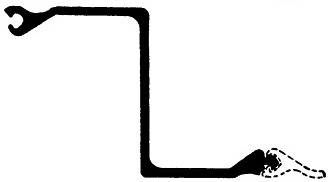


FIG. 7-13*b*.—Z-section Steel Sheet Piling.

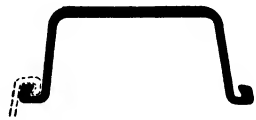


FIG. 7-13*c*.—Larssen Steel Sheet Piling.

the excellent material published by the Carnegie-Illinois Steel Corporation.

7-14. Concrete Sheet Piling. In the construction of piers or wharves where sheet piling forms a part of the permanent structure, reinforced-concrete sheet piles are employed. Sometimes they are rectangular in cross section and are driven in as close contact as possible, the foot being beveled on one edge like timber sheet piles. The larger sizes have tongues and grooves on the edges, the sides of the grooves being splayed so as to engage the tongues more easily. Experience shows that it is not advisable to have a thickness less than 8 in. when a tongue and groove are used, as it is otherwise difficult to obtain the requisite strength for these details. Another plan consists in forming a semicircular groove on both edges, thus forming a cylindrical space with the adjacent pile, to be occupied by the water-jet pipe during sinking and to be filled afterward with grout.

At the terminal piers at Brunswick, Ga., reinforced-concrete sheet piles 18 in. square were used for the bulkhead at the basin. They were 45 ft. long, beveled at the foot to 12 by 18 in., and weighed 7 tons each. Four $\frac{3}{4}$-in. square reinforcing bars extended the full length, and for the lower two-thirds of the length two $1\frac{1}{4}$-in. bars were added in trussed form to help in resisting the maximum bending moment. At the Norfolk Navy Yard the sheet piles were 18 by 24 in. in section, with tongue and groove, and were 55 ft. long. A very extensive use of concrete sheet piling occurred in the Galveston causeway, 9,808 piles being required. They were 10 by 18 in. in section, grooved on both edges, and grouted together after being driven. To improve the protection of the reinforcing rods, it is desirable to place them farther from the surface on the water side than on the other side of a pile.

In order to combine the tensile strength of the interlock for steel sheet piling with the freedom from corrosion of concrete piles, sections have been designed in which a steel pile is cut longitudinally through the web and these halves are cast into a concrete pile on opposite edges. After the piles are driven, the grooves containing the steel interlock are cleaned out with a jet and filled with grout. Another arrangement consists in making a combination pile by enclosing an entire steel sheet pile within a concrete pile, the joints being grouted in a similar manner after driving the piles.

7-15. Driving Steel Sheet Piling. Steel sheet piles are driven generally by steam-hammers, the weights of which are proportioned to that of the piles to be driven. The double-acting steam-hammer is very effective for this purpose because of its rapidity of action, which keeps the pile practically in constant motion, and because it can be handled for this purpose without leads. The lightest hammer of several designs can be handled by one man, and sometimes a step is attached to the hammer frame, so that the weight of the man who operates it may be added while driving. Long sheet piles with heavy sections are driven with heavy steam-hammers and pile drivers.

To protect the head of a steel sheet pile, especially in hard ground, a cap—variously called a "helmet," "follower," or "base attachment"—is generally employed. Its base contains grooves or sockets which fit over the pile. The casting on the right in Fig. 3-8*d* illustrates the base attachment of the Union hammer. This fits into a sleeve, shown in the foreground, which in turn fits into the lower part of the hammer. The base section can rotate in the sleeve, which makes it unnecessary to have the hammer lined up on the exact axis of the piling. The elaborately illustrated catalogues of manufacturers of steam-hammers and of steel sheet piling show plans and sections of caps which are designed for each type of pile. There is usually a transverse as well as a longitudinal groove in the base attachment so as to drive a corner pile or junction pile as well.

In driving through material with a large proportion of sand or clay, the interlocks seal themselves with the material penetrated. Occasionally, strips of wood are driven into the openings, which by swelling help to make the joint watertight above the bottom of the water. Sawdust or wood pulp may also be used to stop leaks. The special rolled sections offer less resistance to driving on account of the absence of rivets or bolt heads; hence they may also be pulled more easily. Under ordinary conditions steel sheet piling may be

pulled and redriven a number of times and finally has considerable scrap value, thus frequently making the cost less than for timber sheet piling which can ordinarily be used only once. Experience has shown instances in which steel sheet piles driven into hard ground could not be used over again, and in exceptional cases it has been impossible to pull it, making it necessary to dredge away some material alongside and to bend it down on the bottom to avoid interference with navigation. Whether this result was due in any measure to improper driving remains uncertain.

Under certain conditions, it is not desirable to drive each pile to its full penetration at one operation. In order to maintain good alignment, or to facilitate closure, it is often advantageous to set up a considerable number of sheet piles and then drive them several feet at a time in succession, repeating the operation until the desired penetration is reached. The same method of driving ahead some distance may be used successfully in avoiding injury to piles when boulders are encountered, if they are not too large. The damage thus becomes local and limited in extent. The water-jet may also be used to aid in displacing boulders. Sheet piles should be carefully handled in transportation, for with a small clearance in the interlock, a bend or kink due to careless handling may cause so much friction that the pile refuses to move on reaching a hard stratum, a condition which may result in crippling the pile if driving is continued.

Steel sheet piling has been successfully driven through submerged logs, old timber cribs, brick, stone, and other debris in made ground. If considerable cribwork or logs have to be penetrated, it may be more economical to construct a special chisel attached to the end of a timber and to cut the timbers with the aid of the chisel and pile hammer before inserting the sheet pile.

The cost of driving steel sheet piling varies between wide limits, depending on the type of soil, depth driven, physical conditions at the site, etc. In placing 3,161 ft. of piling, averaging about 50 ft. long, for a cutoff wall on levee work[1] in Arkansas in 1933, the cost of the piling was 42 cts. and the cost of placing and driving was 12 cts. per square foot of wall. The material penetrated was mostly loess intermixed with sand, silt, and loam. The crews on the pile driving averaged 27½ cts. per hour. The area of piling driven per man-hour was 6.3 sq. ft. and per pile-driver hour was 63 sq. ft. Two sheets were driven at a time, the penetration averaging 1 ft. in 1 to 1½ min. (see also Art. 8-7).

[1] See *Eng. News-Record*, vol. 110, p. 780, June 15, 1933.

7-16. Removing Steel Sheet Piling. Because steel sheet piling is commonly used a number of times and also because, after becoming too badly bent for further use, it still has considerable junk value, special attention has been given the problem of pile pulling. At the present time most pile pulling is done by means of an inverted steam-hammer or by means of a pile extractor, which is merely a direct-acting steam-hammer. These hammers are always used in conjunction with a steady pull maintained on the line, the pull generally not exceeding 25 tons. Sometimes piles are withdrawn

FIG. 7-16*a*.—Inverted Steam-hammer Pulling a Steel Sheet Pile. (*Courtesy of McKiernan-Terry Corp.*)

without the use of hammers. A block and tackle attached to a derrick boom, gallows frame, or gin pole is used, or perhaps a hydraulic jack may be employed.

In 1914 a special machine for pulling sheet piles was developed in England, and at about the same time the ordinary steam-hammer inverted was first used in this country in drawing the sheet piling of the Harlem River section of the New York subway. The rigging consisted of a wire-rope sling suspended from the crane hook and supporting the hammer. A heavy strap of steel passed over the anvil block of the hammer and was fastened to the pulling straps pinned to the pile. This method is illustrated in Fig. 7-16*a*. Figure 7-16*b* shows the type of pile extractor manufactured by the McKier-

nan-Terry Corporation. The chief advantage of the pile extractor is that it can be put into immediate use without any special rigging.

Piles as long as 72 ft. have been pulled with the inverted steam-hammer. The pulling resistance varies between wide limits, depending on the class of material through which the piles are driven and the tightness of the interlock. Where the piling has been overdriven and bent, it may be impossible to pull the same. Except in key piles the pull will usually not exceed 40 tons, whereas a 15-ton derrick equipped with a steam-hammer will develop the

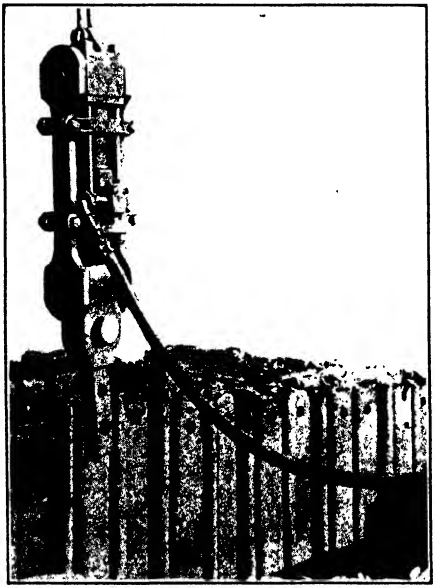


Fig. 7-16*b*.—Pile Extractor Pulling Steel Sheet Piling. (*Courtesy of McKiernan-Terry Corp.*)

equivalent of a steady 75-ton pull. In pulling the key pile of the New York cofferdam described in Art. 8-7, the required pull was 350 tons, which was obtained by the use of block and tackle attached to a gallows frame. The sheet piling of the San Francisco-Oakland bridge were removed by a 125-ton pull exerted by a 16-line tackle of 1-in. cable plus the force developed by the use of a steam-hammer extractor designed for a pull of 25 tons, an offset device making possible the combination pull.

A combination of water-jet and air jet has been successfully used to loosen the material around sheet piling. A mixture of kerosene and heavy oil poured down the interlock may facilitate starting key piles. If the piling has to remain in place a long time, the pulling

may be facilitated by lubricating the joints with graphite or some other material which will prevent corrosion of the interlocking joints. If concrete is deposited next to steel sheet piling, which is to be pulled subsequently, it is essential to prevent contact between the concrete and steel by the use of tar paper or light wooden forms.

Where the pulling is particularly hard, the piling may be cut at river-bed level by means of an acetylene torch operated by a diver. The speed of cutting varies between wide limits, depending on the physical conditions such as depth of water, velocity of current, amount of sediment in the water, etc. The record is 2,118 piles cut in 40 working days by a single crew working a single shift a day. This record of over 50 piles cut per day was made in burning the cofferdam piling used in the pier construction of the Marine Parkway bridge across Jamaica Bay Inlet, Long Island, New York. The piling below pier-top elevation was left in place as a protection against scour. The piling was cut off about 30 ft. below low water at a cost of less than $6 per pile.

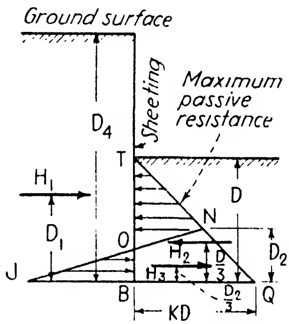


FIG. 7-17*a*. Forces Acting on Sheeting.

7-17. Design of Cantilever Sheet Piling. Where stability of sheet piling depends on embedment of the lower portion, there being no support above the lower ground level as in Fig. 7-17*a*, the problem of design requires finding (*a*) the necessary depth of embedment and (*b*) the required size of piling. The following solution of this type of problem is taken largely from an article by M. A. Drucker entitled Embedment of Piles, Sheeting and Anchor Piles, *Civil Engineering*, vol. 4, p. 622, December, 1934.

In Fig. 7-17*a*, H_1 represents the resultant load from the active earth pressure on a 1-ft. width of the sheeting, and D_1 represents its distance above the foot of the piling. The resisting forces are represented by the triangles *TNO* and *OJB*. According to the beam theory for solid bodies, the resisting forces would be represented by some such line as *JON* produced to meet the surface of the ground. However, in this case the maximum resistance offered by the earth at any elevation cannot exceed the passive earth pressure at that point, hence the distribution of pressure must be as shown. Let K represent the coefficient of passive resistance of the soil, that is, that value which when multiplied by the depth gives

the passive resistance in pounds per square foot at that point; from Art. 2-6, $K = w \dfrac{1 + \sin \phi}{1 - \sin \phi}$

Let H_2 be the pressure represented by the area BTQ acting to the left at a distance $D/3$ above B, and let H_3 be the pressure represented by the area NJQ acting in the opposite direction at a distance $D_2/3$ above B.

To resist horizontal movement,

$$H_1 = H_2 - H_3 = \frac{KD^2}{2} - \frac{D_2 \times JQ}{2}. \qquad (7\text{-}17a)$$

To resist rotation about B,

$$H_1 D_1 = \frac{KD^2}{2} \times \frac{D}{3} - \frac{D_2 \times JQ}{2} \times \frac{D_2}{3}. \qquad (7\text{-}17b)$$

Solving these two equations and eliminating D_2, we have

$$JQ = \frac{(KD^2 - 2H_1)^2}{KD^3 - 6H_1D_1}. \qquad (7\text{-}17c)$$

If we assume that the passive earth resistance JB depends on the distance D rather than on D_4, then $JB = KD$ and $JQ = 2KD$; hence from Eq. 7-17c

$$2KD = \frac{(KD^2 - 2H_1)^2}{KD^3 - 6H_1D_1}. \qquad (7\text{-}17d)$$

For known values of H_1, D_1, and K, this equation can be solved for D by assuming values of the latter until the equation is satisfied.

Where there is no load on the surface of the ground and the soil is homogeneous for the entire depth, $H_1 = \frac{1}{2}wCD_4^2$, where $C = \dfrac{1 - \sin \phi}{1 + \sin \phi} = \dfrac{w}{K}$, and $D_1 = D_4/3$; then Eq. 7-17d becomes

$$2\frac{w}{C}D = \frac{\left(\frac{w}{C}D^2 - wCD_4^2\right)^2}{\frac{w}{C}D^3 - wCD_4^3},$$

which, when simplified, becomes

$$C^4D_4^4 + 2C^2(D_4^3D - D_4^2D^2) - D^4 = 0,$$

or

$$C^4 + 2C^2\left[\frac{D}{D_4} - \left(\frac{D}{D_4}\right)^2\right] - \left(\frac{D}{D_4}\right)^4 = 0. \qquad (7\text{-}17e)$$

From this equation the diagram of Fig. 7-17*b* was obtained by substituting values of D/D_4 and solving for C.

As an example, let it be required to design the sheet piling for a cantilever height, $D_4 - D$, of 7.5 ft., where $\phi = 34$ deg. and $w = 100$ lb. per cu. ft. The value of $C = \dfrac{1 - \sin\phi}{1 + \sin\phi} = \dfrac{1 - 0.559}{1 + 0.559}$ $= 0.283$. Assume $D_4 = 14$ ft., then from Fig. 7-17*b*, $D/D_4 = 0.46$ and $D = 0.46 \times 14 = 6.44$, say 6.5 ft. Therefore $D_4 - D = 14 - 6.5 = 7.5$ ft.; hence the correct value of D_4 was assumed.

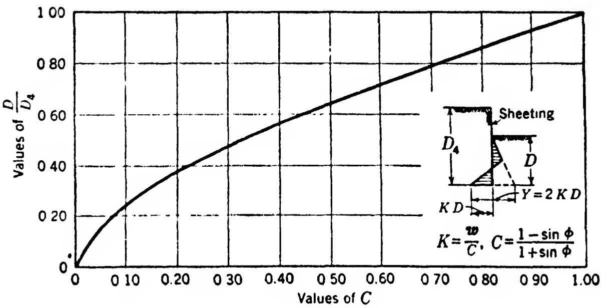


FIG. 7-17*b*.—Diagram for Embedment of Sheeting.

The distance y below the point T (Fig. 7-17*a*) at which the shear is zero, and hence the moment a maximum, is found from the equation

$$\frac{1}{2} \times 100 \times 0.283(7.5 + y)^2 = \frac{1}{2} \times 100 \times \frac{1}{0.283} y^2, \qquad \text{or} \qquad y = 2.96 \text{ ft.}$$

Hence

$$M = \frac{1}{6} \times 100 \times 0.283 \times \overline{10.46}^3 - \frac{1}{6} \times 100 \times \frac{1}{0.283} \times \overline{2.96}^3$$
$$= 3{,}880 \text{ ft.-lb. per ft. of wall.}$$

If an allowable fiber stress of 18,000 lb. per sq. in. is used, the required sectional modulus for steel piling is $3{,}880 \times 12/18{,}000 = 2.59$ in.^{3} per foot of wall. A straight web interlocking pile 15 in. wide and weighing 38.8 lb. per lin. ft. is available having a sectional modulus of 3.0 in.^{3} per foot of wall.

For wooden Wakefield sheet piling, when an allowable fiber stress of 1,200 lb. per sq. in. is used, the required sectional modulus is $3{,}880 \times 12/1{,}200 = 38.8$ in.^{3} per ft. of wall. Three thicknesses of 3 in., dressed down to $2\frac{5}{8}$ in., give a sectional modulus of

$$\frac{3 \times 12 \times \overline{2.62}^2}{6} = 41.1 \text{ in.}^3 \text{ per ft. of wall.}$$

7-18. Design of Anchored Bulkheads. Figure 7-18*a* represents a typical anchored steel sheet-piling bulkhead in which the reaction of the upper end of the sheet piling is taken by anchor rods and an anchor system and that of the lower end by passive earth pressure. The anchor system and tie rods are usually placed

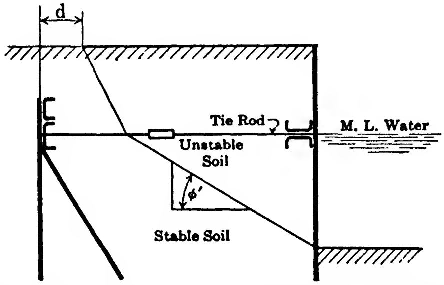


FIG. 7-18*a*.—Anchored Steel Sheet-piling Bulkhead.

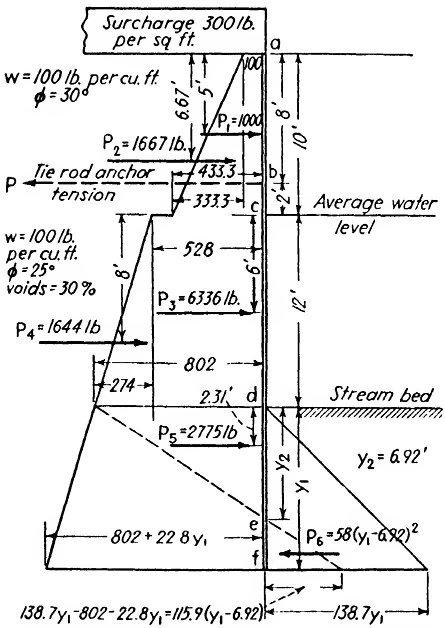


FIG. 7-18*b*.—Forces Acting on Anchored Sheet-piling Bulkhead.

near the water line to preserve them against rust and rot, as well as to decrease the maximum moment in the sheet piling. However, they are not often placed below water level, owing to difficulties involved in the construction work. In designing the bulkhead, we must find (*a*) the depth to which the sheet piling must be driven, (*b*) the stress in the tie rods, and (*c*) the maximum moment in the

sheet piling. With this information we can determine the length and size of the sheet piling, the size of the waling pieces, and the make-up of the tie rods and anchorage system.

A very complete analysis of this type of problem will be found in the literature[1] referred to in the last paragraph of Art. 7-13. An approximate solution of this type of design is as follows: Let it be required to determine the depth of penetration y_1 of the sheet piling of Fig. 7-18*b* to prevent movement of the toe. Let it also be required to find the stress in the tie rod and the maximum moment in the sheet piling. The weights of materials and values of coefficients of friction are given on the figure. Below the saturation level the loss of weight of the soil equals the weight of the displaced water; hence the weight below water level is $100 - 62.4 \times 0.70 = 56.3$ lb. per cu. ft.

If Eq. 2-6*a* is used, the intensities of pressure on the back of the wall are as follows:

At *a*

$$300\,\frac{1 - \sin 30 \text{ deg.}}{1 + \sin 30 \text{ deg.}} = 300\,\frac{1 - 0.50}{1 + 0.50} = 100 \text{ lb. per sq. ft.}$$

Just above *c*

$$(300 + 10 \times 100)\,\frac{1 - \sin 30 \text{ deg.}}{1 + \sin 30 \text{ deg.}} = 1{,}300\,\frac{1 - 0.50}{1 + 0.50}$$
$$= 433.3 \text{ lb. per sq. ft.}$$

Just below *c*

$$(300 + 10 \times 100)\,\frac{1 - \sin 25 \text{ deg.}}{1 + \sin 25 \text{ deg.}} = 1{,}300\,\frac{1 - 0.4226}{1 + 0.4226}$$
$$= 528 \text{ lb. per sq. ft.}$$

At *d*

$$(300 + 10 \times 100 + 12 \times 56.3)\,\frac{1 - \sin 25 \text{ deg.}}{1 + \sin 25 \text{ deg.}}$$
$$= 1{,}976\,\frac{1 - 0.4226}{1 + 0.4226} = 802 \text{ lb. per sq. ft.}$$

At *f*

$$(300 + 10 \times 100 + 12 \times 56.3 + y_1 \times 56.3)\,\frac{1 - \sin 25 \text{ deg.}}{1 + \sin 25 \text{ deg.}}$$
$$= (802 + 22.8y_1) \text{ lb. per sq. ft.}$$

If Eq. 2-6*b* is used, the intensities of passive earth pressure on the front of the wall are as follows:

[1] See also *Civil Eng.*, vol. 3, p. 615, November, 1933; vol. 4, p. 197, April, 1934.

At d: 0.
At f

$$y_1 \times 56.3 \frac{1 + \sin 25 \text{ deg.}}{1 - \sin 25 \text{ deg.}} = y_1 \times 56.3 \frac{1 + 0.4226}{1 - 0.4226} = 138.7 y_1$$
lb. per sq. ft.

The intensities, as well as the total pressures per foot of wall, are shown on the figure, the trapezoids which graphically represent these pressures being divided into rectangles and triangles of pressure. The dotted line below ground level represents the combined active and passive pressures. The pressures are as follows:

$P_1 = 100 \times 10 = 1{,}000$ lb. $\quad P_2 = \frac{1}{2} \times 333.3 \times 10 = 1{,}667$ lb.
$P_3 = 528 \times 12 = 6{,}336$ lb. $\quad P_4 = \frac{1}{2} \times 274 \times 12 = 1{,}644$ lb.

To locate the point e where the intensity of earth pressure is zero, we have

$$\frac{y_2}{y_1 - y_2} = \frac{802}{115.9 y_1 - 802} \quad \text{or} \quad y_2 = 6.92 \text{ ft.}$$

$P_5 = \frac{1}{2} \times 802 \times 6.92 = 2{,}775$ lb. $\quad P_6 = \frac{1}{2} \times 115.9(y_1 - 6.92)^2 = 58(y_1 - 6.92)^2.$

To find the necessary depth of penetration, y_1, we take $\Sigma M_b = 0$; $+ 1{,}000 \times 3 + 1{,}667 \times 1.33 - 6{,}336 \times 8 - 1{,}644 \times 10 - 2{,}775 \times 16.31 + 58 \times (y_1 - 6.92)^2 \times [20.92 + \frac{2}{3}(y_1 - 6.92)] = 0$;

$$58(y_1 - 6.92)^2(0.667 y_1 + 16.3) = 107{,}137;$$
$$(y_1 - 6.92)^2(y_1 + 24.45) = 2{,}774.$$

If we solve this by trial, $y_1 = 15.3$ ft.

To find the stress P in the tie rod, we use $\Sigma x = 0$; $+ 1{,}000 + 1{,}667 + 6{,}336 + 1{,}644 + 2{,}775 - 58(15.3 - 6.92)^2 - P = 0$ or $P = 9{,}350$ lb. per ft. of wall.

When the rods are spaced at 7 ft. center to center, the total load on each rod will be $9{,}350 \times 7 = 65{,}450$ lb. This requires the use of a $2\frac{1}{4}$-in. diameter upset rod for an allowable tensile stress of 18,000 lb. per sq. in.

The load will be transferred to the tie rods through waling pieces consisting of a pair of channels riveted to the sheet piling. Assuming these channels to act as simple beams under uniform load, the maximum moment is $\frac{1}{8} \times 9{,}350 \times 7^2 = 57{,}270$ ft.-lb. Using the same unit stress in bending as specified above for tension, the required sectional modulus is $57{,}270 \times 12/18{,}000 = 38.2$ in.^{3} Two 12-in. 20.7-lb. channels furnish a sectional modulus of 42.8 in.^{3}

The maximum moment in the sheet piling occurs where the shear equals zero. Denoting the distance from c to this point as y, we have

$$-9{,}350 + 1{,}000 + 1{,}667 + 528y + \frac{1}{2} \times y \times \frac{y}{12} \times 274 = 0;$$
$$11.42y^2 + 528y = 6{,}683;$$
$$y = 10.4 \text{ ft.}$$

Hence the maximum moment is

$$9{,}350 \times 12.4 - 1{,}000 \times 15.4 - 1{,}667 \times 13.73 - \tfrac{1}{2} \times 528 \times \overline{10.4}^2 - \tfrac{1}{6} \times \tfrac{274}{12} \times \overline{10.4}^3 = 44{,}800 \text{ ft.-lb. per ft. of wall.}$$

This gives a required sectional modulus of $44{,}800 \times 12/18{,}000 = 29.87$ in.^{3} per ft. of wall. A Z-section is rolled that is 21 in. wide, weighs 32 lb. per sq. ft. of wall, and has a sectional modulus of 38.3 in.^{3} per ft. of wall.

The location of the anchor system should be in stable ground, well back of the sheet-piling wall. The distance d of Fig. 7-18a should preferably be at least 10 ft., this being measured from the edge of the unstable soil. The angle of repose of the fill locates the planes separating the stable from the unstable material, the angle of repose above water line being ϕ for dry soil and the angle ϕ' for saturated soil. The design of the anchorage piles is largely a matter of judgment.

7-19. Design of Gravity Bulkheads. A steel sheet-piling bulkhead of the gravity type consists of two parallel walls of piling tied to each other, with the space between the walls filled with soil or broken stones. Here stability is furnished by the weight of the confined material instead of by anchorage piles as in Art. 7-18. The following analysis of the problem is largely taken from an excellent article[1] by Raymond P. Pennoyer entitled Gravity Bulkheads and Cellular Cofferdams, published in *Civil Engineering*, vol. 4, p. 301, June, 1934.

In designing a bulkhead of this type we determine the following: (a) the required width to resist overturning, (b) the factor of safety against sliding, (c) the internal shear in the confined material, and (d) the stresses in the component parts of the structure. As an example let us consider the bulkhead of Fig. 7-19a taken from the article referred to above. The forces on the back of the structure are

[1] See also bulletin entitled "Carnegie Steel Sheet Piling," Carnegie Steel Company, 1931.

$$P_1 = \frac{1}{2} w_e \frac{1 - \sin \phi}{1 + \sin \phi} h'^2$$

$$P_2 = w_e \frac{1 - \sin \phi}{1 + \sin \phi} h'h''$$

$$P_3 = \frac{1}{2} w_{e \text{ in } w} \frac{1 - \sin \phi'}{1 + \sin \phi'} h''^2$$

and the total pressure is

$$P = P_1 + P_2 + P_3.$$

In these formulas

w_e = weight of dry earth in pounds per cubic foot
$w_{e \text{ in } w}$ = weight of submerged earth in pounds per cubic foot
ϕ = angle of repose of dry earth in degrees
ϕ' = angle of repose of submerged earth in degrees

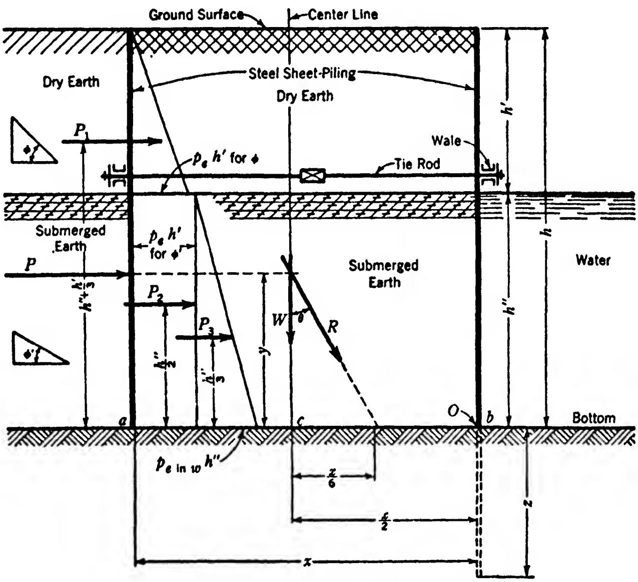


FIG. 7-19a.—Gravity Bulkhead.

The distance y from the bottom to the line of action of P is given by the formula

$$y = \frac{P_1\left(h'' + \frac{h'}{3}\right) + P_2 \frac{h''}{2} + P_3 \frac{h''}{3}}{P}.$$

The resultant of the lateral force P and the weight of the fill $W = x(w_e h' + w_{e \text{ in } w} h'')$ should not lie outside the point $x/6$ from

the center of the base, otherwise there will be tension or uplift at the heel (point a) of the bulkhead. Therefore,

$$\tan \theta = \frac{x/6}{y} = \frac{P}{W}$$

$$x = \frac{6Py}{W} = \sqrt{\frac{6Py}{w_e h' + w_{e \text{ in } w} h''}}.$$

The base should be taken at some distance below the bottom, unless rock is present. This distance will depend on the stability and passive earth pressure of the soil.

The force tending to slide the bulkhead on the base is P, while the resistance equals the friction of the fill on the base plus the toe resistance of the piling. Where the surface is bedrock into which piling cannot be driven, the structure is safe against sliding as long as the tangent of the angle θ of Fig. 7-19a is less than the coefficient of friction of the soil on the rock. This coefficient is seldom less than 0.5. Where the bulkhead is built on soil, the piling will usually be driven some distance into the bed.

Applying the law of mechanics governing internal shear in beams, the maximum intensity of horizontal and vertical shear occurs at the point c of Fig. 7-19a and equals $\frac{3}{2}P/x$. The resistance intensity, which must exceed this, equals $(w_e h' + w_{e \text{ in } w} h'') \tan \phi'$.

The design of the outer wall and of the rods is similar to that given in the preceding article. Where the plane ab is rock, a trench may be cut in the rock to hold the toe of this wall in place, or holes may be blasted at intervals in which certain of the piles are placed. Another method sometimes used is to connect with tie rods the bottom of alternate piles to corresponding piles in the back wall and to drive the pairs simultaneously. The back wall is much less heavily stressed than the front wall, but any theoretical analysis is highly uncertain.

CHAPTER VIII

COFFERDAMS

8-1. The Cofferdam Process. When, for some purpose, it is desired to exclude the water and expose a portion of the bottom of a river, lake, or other body of water, a structure called a cofferdam is employed. This cofferdam is a temporary structure, practically watertight, and large enough to provide adequate room for working.

Defined, a cofferdam is a temporary structure used for excluding the water from a given site, or area, either wholly or to such a degree that, with a reasonable amount of pumping, the permanent substructure may be built within it in the open air, or other work may be accomplished.

The building of the permanent substructure may include pile driving, placing grillages, building piers and abutments, etc., while other work may include the construction of dams, removal of sunken vessels, etc. Where the ground is saturated with water, cofferdams are sometimes used in placing foundations for buildings.

Cofferdams are usually built in place. They may be self-contained or may depend for strength on the natural bottom, as is the case where guide piles are used. Bracing is often used to resist the lateral pressure against the walls.

To obtain watertightness, the sides of the cofferdam must be tight and the soil on which the cofferdam rests must be impervious. If the latter condition does not exist, either the sides of the cofferdam must extend through the pervious material to an impervious stratum or else a layer of concrete must be spread over the bottom inside the cofferdam and allowed to harden before pumping is begun. Absolute watertightness is seldom sought, it being cheaper to pump a moderate amount of leakage than go to the heavy expense of building a structure that will not leak at all. The cofferdam should be so designed that the combined cost of construction, maintenance, and pumping will be a minimum.

At the present time (1941) the cofferdam process is being used for depths far greater than was thought practicable a few years ago when 30 to 40 ft. was considered to be about the economical limit. For example, a head of 100 ft. will obtain in the round cellular

cofferdam of the Kentucky (formerly Gilbertsville) Dam Project.[1] At Grand Coulee a section of cofferdam was 90 ft. high, and a cofferdam of similar design was 85 ft. high on the Delaware River aqueduct project at the Kensico Reservoir in New York. Among the boldest plans for cofferdam construction ever used were those of the New Jersey tower of the George Washington bridge (Art. 8-6). Here two cofferdams, each $98\frac{1}{2}$ by 109 ft. in plan, successfully withstood a head of 79 ft. of water.

In placing the foundations of pier W2 of the San Francisco-Oakland bridge, a single-wall steel sheet-pile cofferdam, $52 \times 121\frac{1}{3}$ ft. in plan, braced with guide piles, walings, and internal bracing, was used where the average depth from water level to rock was 88 ft. and the maximum depth 101 ft. In building the south pier of the Golden Gate bridge, a concrete fender ring was constructed to act as a cofferdam, the depth of water being 106 ft. at high tide. In building a bridge across Lake Champlain between Crown Point and Chimney Point (Art. 8-6), single-wall cofferdams 22 by 52 ft. in plan were driven to a depth of $96\frac{1}{2}$ ft. below water level. However, in all three cases noted in this paragraph, a considerable height of concrete was placed through the water and allowed to harden before unwatering the structure, thus considerably reducing the heads sustained by the cofferdams.

Cofferdams may be built of earth, timber, steel, or concrete. They may be divided into the following classes; earth, sheet piles on guide piles, sheet piles on frames, sheet piles on cribs, cellular, and movable.

8-2. Earth Cofferdams. Of the six classes the earth cofferdam is the oldest in origin and simplest in construction. Its use is usually limited to shallow water with low velocities of current. It is made of a bank of earth placed around the site to be enclosed, the earth bank being of a thickness sufficient to furnish the required stability and to keep the leakage down to a small amount. The earth bank should be carried up 2 or 3 ft. above the water level, with a width of at least 3 ft. at the top and with side slopes corresponding to the natural slope of the material. The embankment should preferably be composed of a mixture of clay and sand or gravel, but, if clay is scarce, the bank may be composed of sand with a clay wall in the center.

The amount of embankment may be somewhat reduced by using one or two rows of sheet piling, in which case the cofferdam may resemble more or less closely the sheet-pile cofferdam described

[1] *Civil Eng.*, vol. 9, p. 551, September, 1939.

in later articles. As to whether in any given case the cofferdam should be classed as an earth or sheet-pile cofferdam will depend upon whether or not stability and watertightness depend primarily upon the earth filling.

Where the depth of water is not more than 4 or 5 ft. and the velocity of the current would wash away loose material, cofferdams may be made of ordinary canvas bags about half-filled with a mix-

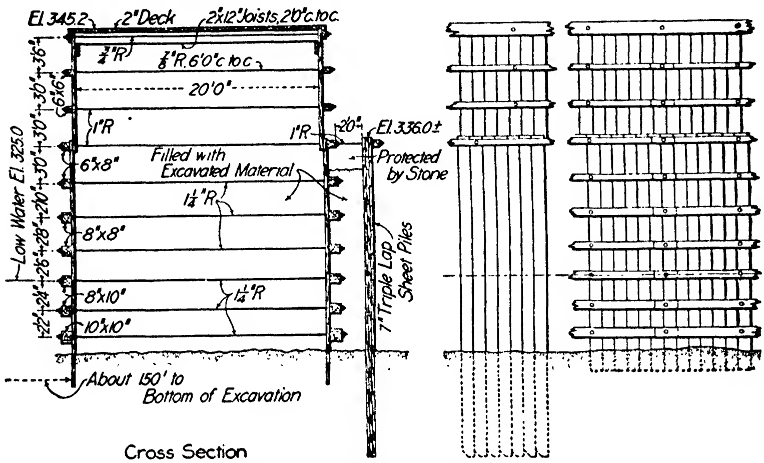


Fig. 8-2*a*.—Sheeting for Earth Cofferdam on the Ohio River.

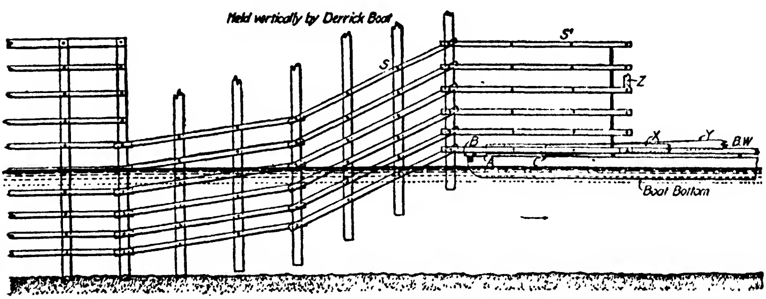


Fig. 8-2*b*.—Method of Constructing Ohio River Cofferdam.

ture of clay and sand. It is important that the bags shall be only partially filled, for otherwise they will not pack together closely.

Figure 8-2*a* illustrates the cross section of the earth cofferdam with sheeting used in the construction of the Ohio River lock and dam 48, where the bottom was composed of sand. To break the current, a line of sheet piling was first driven. Frames were then placed by a boat, as shown in Fig. 8-2*b*, and connected to the sheet piling. The frame was built by moving the boat to the right until

A cleared *B*, then plank *X* was raised to a vertical position, and the rods were placed. This operation was then repeated for plank *Y*. Before raising plank *Z*, the lower wale *BW* was fastened, after which the section *S* was allowed to drop to the bottom and the lifting line transferred from *S* to *S'*. Vertical planking was placed against the frames and the interior then filled with dredged material. Gravel was placed along the outside of the sheet piling up to its top and on a slope of about 45 deg. The space between the sheet piling and sheeting was also filled with earth, and finally sand was placed against the inside wall of sheeting up to the elevation of the sheet-piling top. This sand had a very gentle slope, running approximately 100 ft. before reaching the elevation of the bottom of the sheeting.

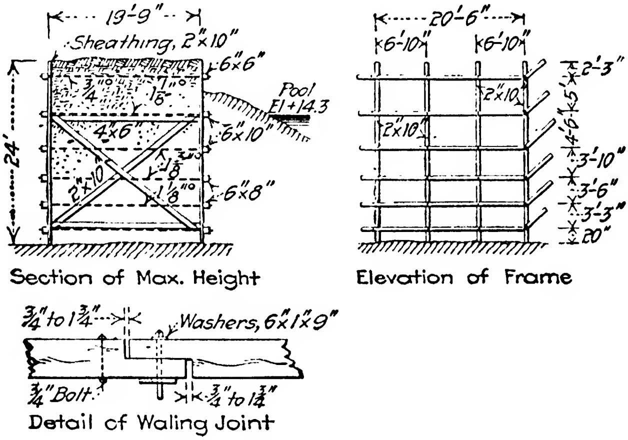


FIG. 8-2c.—Frame for Earth Cofferdam.

In another cofferdam of this type (Fig. 8-2*c*) the method adopted consisted in laying horizontally and loosely the two separate sets of walings for a $20\frac{1}{2}$-ft. section on two corresponding frames or cradles lying opposite each other on the deck of the working barge. Each cradle was provided with projecting arms bolted on at distances corresponding to the spacing of the walings and supporting the latter when the cradles were set up. The two cradles were then lifted to a vertical position, the rods attached (as well as struts at top and bottom), and connection made to the preceding section. The frames were then lifted slightly, allowing the cradles to clear and be lowered, after which the scow was moved so as to launch the framework.

8-3. Sheet Piling Supported by Guide Piles. In the simpler types of cofferdams, particularly where wood sheet piling is used,

the chief function of the sheet piling is to give watertightness to the structure. To this end some form of intermeshing or interlocking piling is always used. Strength to resist the pressure of the water outside is furnished by guide piles, frames, or cribs, together with internal bracing in many cases.

The use of guide piles and wales makes an economical type of construction where the piles can be driven some distance into the soil. For large cofferdams where it is not economical to use internal

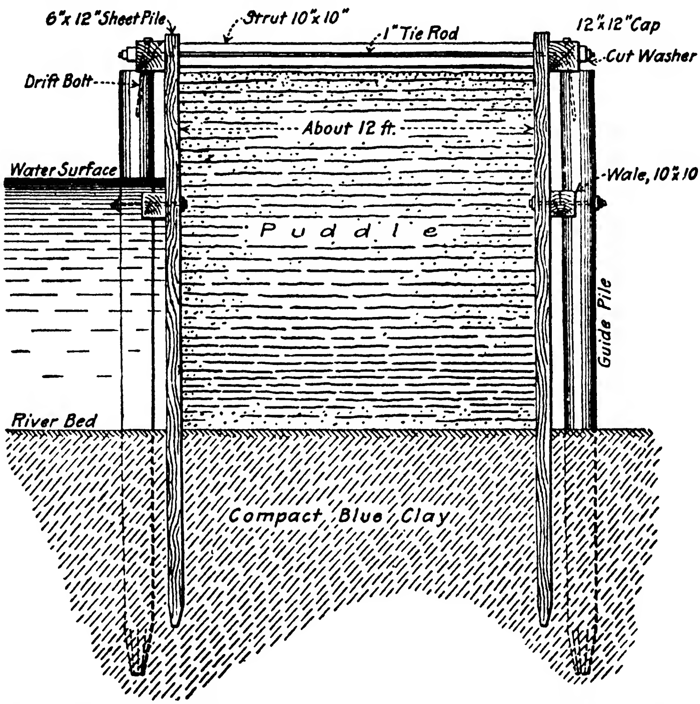
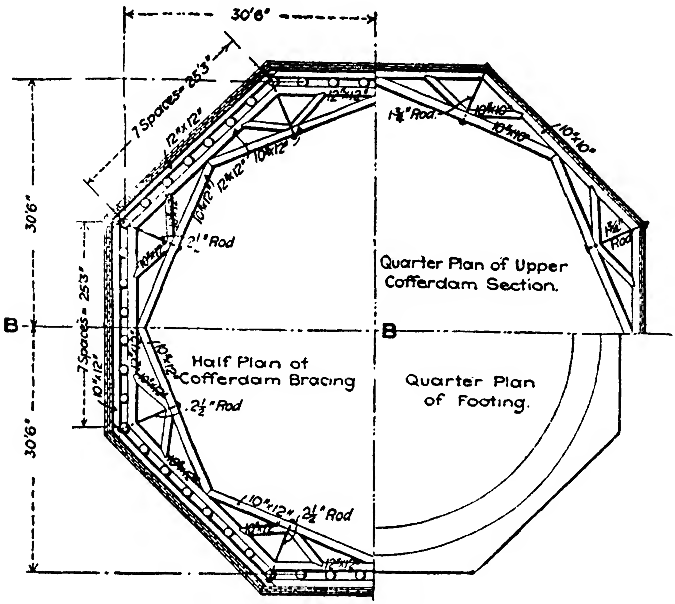


FIG. 8-3*a*.—Section of the Double Wall of a Cofferdam Showing Puddle Chamber.

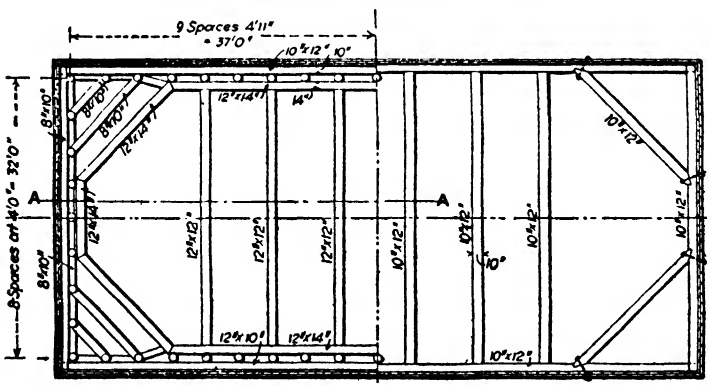
bracing, the double-wall type is favored. As shown in Fig. 8-3*a*, the double-wall type is composed of vertical guide piles, horizontal wales and cap timbers, vertical sheet piles, and a puddle filling. Rods are usually put in near the top to connect each pair of guide piles in order to prevent the filling from spreading the walls apart. Struts are often placed near and parallel to the tie rods, serving to hold the two walls apart.

The guide piles are driven deeper into the earth than the sheet piles, the aim being to drive them far enough to develop the full transverse strength of the pile when acting as a free cantilever

above the ground. The sheet piling should be driven to a fairly impervious stratum to prevent leakage into the cofferdam from below. The space between the walls is filled with earth, preferably



Cofferdam for Center Pier No. 5.



Half Plan of Lower Cofferdam. Half Plan of Upper Cofferdam.

Cofferdam for Piers 4, 5 and 6

FIG. 8-36.—Cofferdams with Single Walls of Timber Sheet Piling Supported by Wales and Guide Piles.

an intimate mixture of sand and clay, or gravel and clay, to form a puddle (Art. 8-9), which will materially assist the sheet piling in making the cofferdam watertight. This puddle should be placed

in thin layers and thoroughly tamped in a damp state. Before placing the same, it will usually be advisable to dredge out the soft material on the bottom to an impermeable stratum. This puddle filling, in addition to promoting watertightness, will materially strengthen the structure. Clay is often banked around the outside of the cofferdam to further safeguard it against leakage.

The cofferdam should be wide enough to develop the required stability, furnish watertightness, and afford sufficient space for

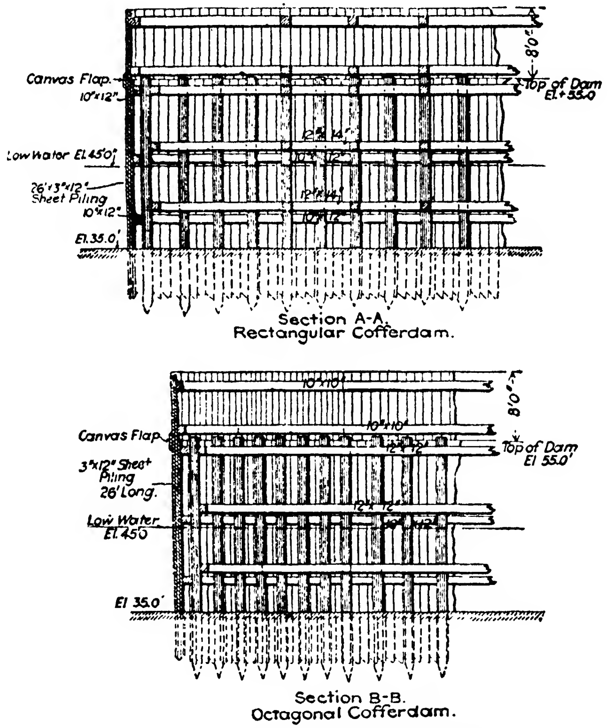


FIG. 8-3c.—Elevation of Cofferdam Walls.

placing machinery, gangways, etc. One rule for unbraced cofferdams is to make the width equal to the height above the ground up to 10 ft., and for greater heights to make it 10 ft. plus one-third the height in excess of 10 ft.

Where the space available for the cofferdam is restricted, the single-wall type may be used, stability being obtained by the use of internal bracing. Figures 8-3*b* and 8-3*c* show the details of typical rectangular and octagonal cofferdams of moderate size. Before placing these cofferdams, the bed of the river was dredged

of soft material. Cofferdam guide piles were then driven and 10- by 12-in. wales bolted to the outside, after which 9- by 12-in. triple-lap sheet piling was driven against the latter, penetrating the earth

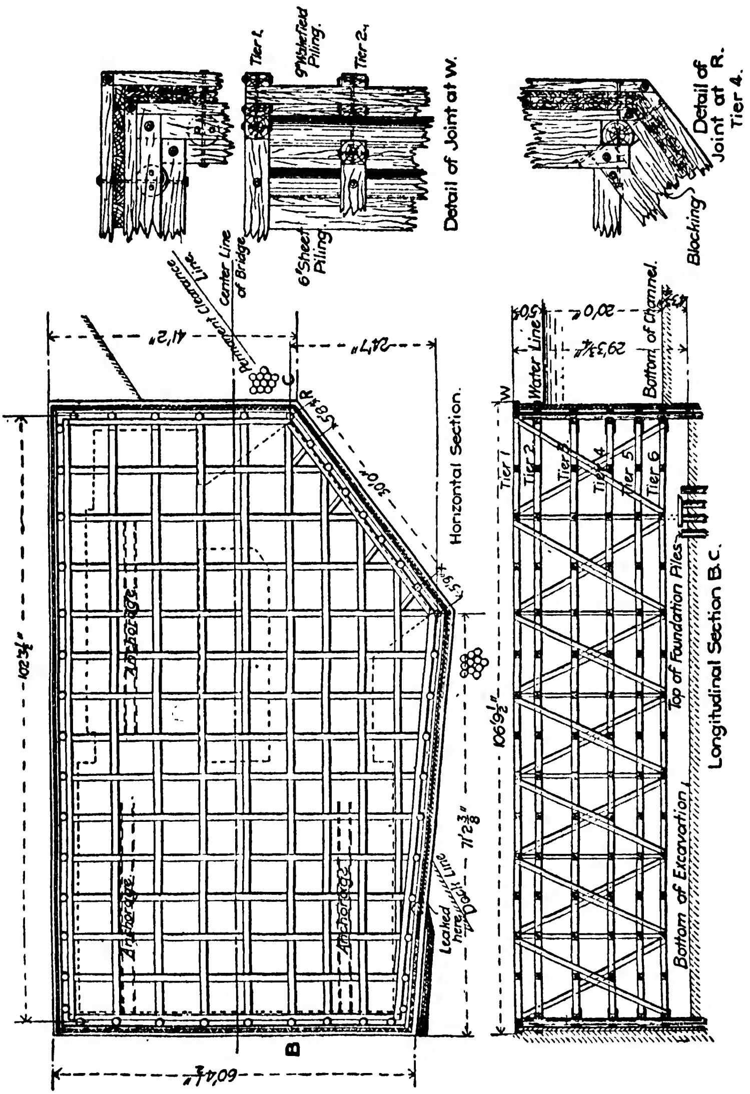


Fig. 8-3d.—Cofferdam for Substructure of Rolling Lift Bridge at Taylor Street, Chicago.

from 4 to 6 ft. The octagonal cofferdam was braced by annular trusses, which, by their archlike action, proved to be a very rigid form of bracing and yet offered no obstruction to work being done inside the cofferdam.

A good example of a large single-wall cofferdam, strongly braced, is illustrated in Fig. 8-3*d*, the structure being used to found the pier of a lift bridge for the Chicago Terminal Transfer Railroad. Two sides of the cofferdam were on land, one in water, and the other two partly in water and partly on land. A row of guide piles, from 6 to 8 ft. apart and 40 ft. long, were first driven. Six tiers of inside 12- by 12-in. and outside 6- by 12-in. waling pieces were bolted to the piles. On the land side 6- by 12-in. sheet piles were driven

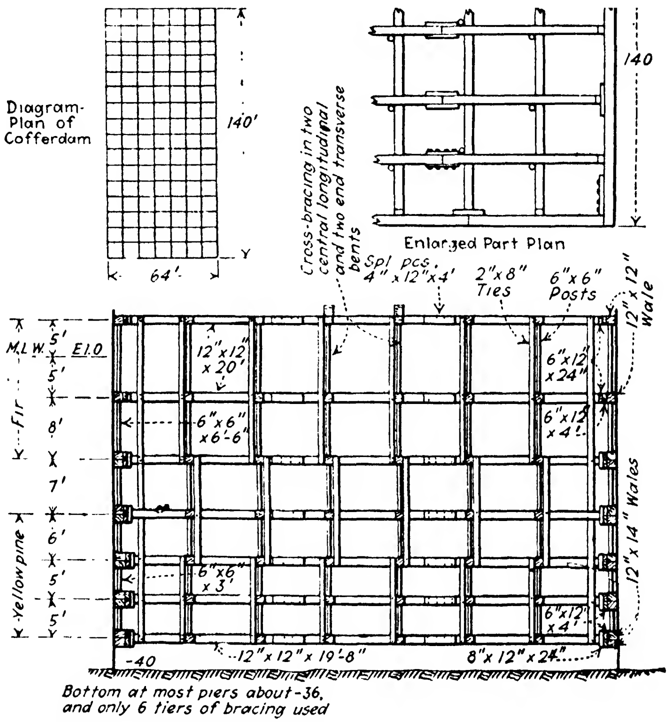


FIG. 8-3*e*.—Bracing of Cofferdam for Arlington Memorial Bridge.

between the outer wales and 6- by 12-in. horizontal guide pieces at the surface of the ground and 4 ft. below it. On the water sides Wakefield sheet piling, made of three thicknesses of 3- by 12-in. planks, was used. The transverse and longitudinal bracing consisted of 12- by 12-in. timbers spaced from 6 to 8 ft. apart horizontally and from 4 to 6 ft. apart vertically. At alternate intersections these braces were supported on 8- by 8-in. timbers, which, together with 2- by 10-in. diagonal bracing, served to truss the six tiers of bracing on four of the longitudinal lines and on six of the transverse lines.

In placing single-wall cofferdams in which to build the piers and abutments of the Arlington Memorial bridge over the Potomac River, steel sheet piling was used. The largest cofferdam was 64 by 140 ft. in plan and about 40 ft. high. Guide piles, braced by batter piles, were driven at 8-ft. centers and connected with three lines of walers on the inside to form guides for the steel sheet piling. The sheet piling was then set in place in the mud, and at intervals a pile was driven to rock and bolted to a waler to hold the wall of sheeting. After closure, the piles were driven to rock.

The inside was excavated to rock by a clam-shell bucket prior to placing the internal bracing, a partial set of bracing being framed at the surface to steady the piling while excavation was carried on. On completion of excavation, the internal bracing system shown in Fig. 8-3*e* was placed. In this illustration the guide piles and three rows of walers used to guide the piling in driving are not shown. The longitudinal and transverse horizontal bracing struts in each tier were spaced on 9-ft. centers. The successive tiers of the cribbing were framed at the water surface inside of the cofferdam, a clearance of 1 to 2 in. being allowed on each side. As the cribbing was built up, it was sunk, sand ballast being used to overcome the buoyancy.

8-4. Sheet Piling on Wooden Frames. Where the nature of the bottom is such that guide piles cannot be driven or where the depth is too great to make the use of guide piles economical, a frame may be used to guide and support the sheet piling. For small or medium-sized cofferdams in water and for land cofferdams, wooden frames are satisfactory, but for large or deep cofferdams, structural-steel bracing is generally employed. Steel sheet piling is universally used for the larger structures.

Figure 8-4*a* illustrates the use of wooden frames, which are usually built on the shore, floated to the site, and sunk by weighting with steel rails or other heavy material, after which the piling is placed. The cofferdam illustrated had V-shaped ends to diminish the force of the current against the structure and was held in place by wire guys anchored to rocks on the sides of the river. The covering consisted of 9- by 12-in. Wakefield sheet piling. In driving this piling, care was taken to broom the lower ends to give a close fit to the irregular rock surface.

To aid in giving watertightness to the structure, canvas was placed around the outside of the cofferdam and was so arranged that the lower part rested flat on the river bed for a distance of 8 ft. out from the dam, while the upper part extended above water level. The lower part of the canvas was first weighted down with iron rails

and sandbags to make it fit closely, after which about 50 carloads of gravel were placed on it. As the water was pumped out, the cofferdam was thoroughly braced as shown, but on building the pier, this bracing was removed and the cofferdam walls braced against the pier.

Wooden frames were used to brace the land cofferdams employed in placing some of the piers of the Tunkhannock Viaduct of the

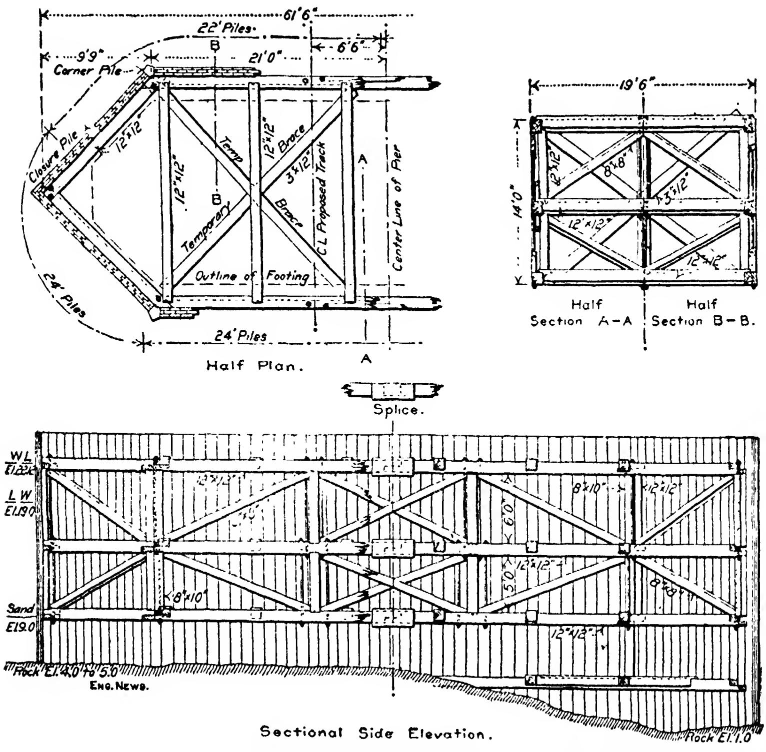


FIG. 8-4a.—Cofferdam for Pier of Chicago, Milwaukee, and St. Paul Railway, Kilbourn, Wis.

Delaware, Lackawanna & Western Railroad, which had a maximum depth of nearly 100 ft., and a depth of 65 ft. below ground-water level. The cofferdam was commenced by assembling on the surface of the ground a 43- by 49-ft. rectangle made of 12- by 12-in. horizontal timbers spliced together to form one course of inner wales. Vertical posts were set up on this course and supported a second similar course about 16 ft. above it; two corresponding courses of exterior wales were erected outside of these and about 6 in. in the clear from them.

Lackawanna steel sheet-pile units 30 ft. long were then placed between the outer and inner wales and driven by a steam-hammer going round and round the cofferdam, driving each pile unit 2 or 3 ft. at a time. As the piling was driven, the interior was excavated and the cofferdam braced with successive tiers of 12- by 12-in. longitudinal and transverse struts.

When this set of piling was driven to its full length, an exterior row, concentric with the inner row and 4 ft. 8 in. beyond the same, was assembled and first driven to a penetration of about 12 to 15 ft. The space between the two rows was excavated, and at the same time the inner row continued to be driven, the upper tiers of bracing of the latter being transferred to the bottom and new sets of bracing furnished to the outer piling. In this way, by driving both outer and inner rows to their required positions, the excavation was carried to rock. The advantage of the two rows of piling was in the easier driving thereby obtained.

8-5. Deep Cofferdams Braced with Steel. Considerable progress has been made in recent years in the design and construction of deep cofferdams composed of interlocking steel sheet piling, either in the form of single walls braced with steel frames or in a cellular form. The most difficult problem is placing the bracing frame in position, not only because of its considerable weight, but also on account of the work having to be done through water prior to pumping out the cofferdam.

Figure 8-5*a* shows deep 20- by 52-ft. all-metal cofferdams used in placing piers 6 and 7 of the Lake Champlain bridge at Crown Point State Park, N. Y. Here bedrock, on which the piers were founded, was 96.5 ft. below low-water level, low water being at elevation 92.5 and rock at elevation −4. The sites were first dredged to elevation +50, which required taking out about 18 ft. of material at one site and practically none at the other. Following this, for each cofferdam, a single row of arched steel sheet piling 98 ft. long—58- and 40-ft. sections spliced—and weighing 25 lb. per sq. ft. of wall, was driven inside a timber guide frame supported by timber bearing piles from 60 to 80 ft. long.

The bracing for the lower portion of the cofferdams, consisting of four horizontal tiers of framed I-beam walers and cross struts, was placed on the lake bottom within the cofferdams by derricks prior to excavating. The bracing cages above elevation 50 were then placed. These consisted of horizontal frames composed of I-beam walers and struts, the horizontal frames being connected by means of vertical and diagonal angle irons. The bracing cages

were handled by two derricks and were hung from timbers placed across the tops of the cofferdams.

The soil within the cofferdams was excavated by dredging through the water with clamshell buckets. Because of the presence of the bracing walers the buckets could not dig close to the walls and consequently a high-pressure jet, handled by divers was used

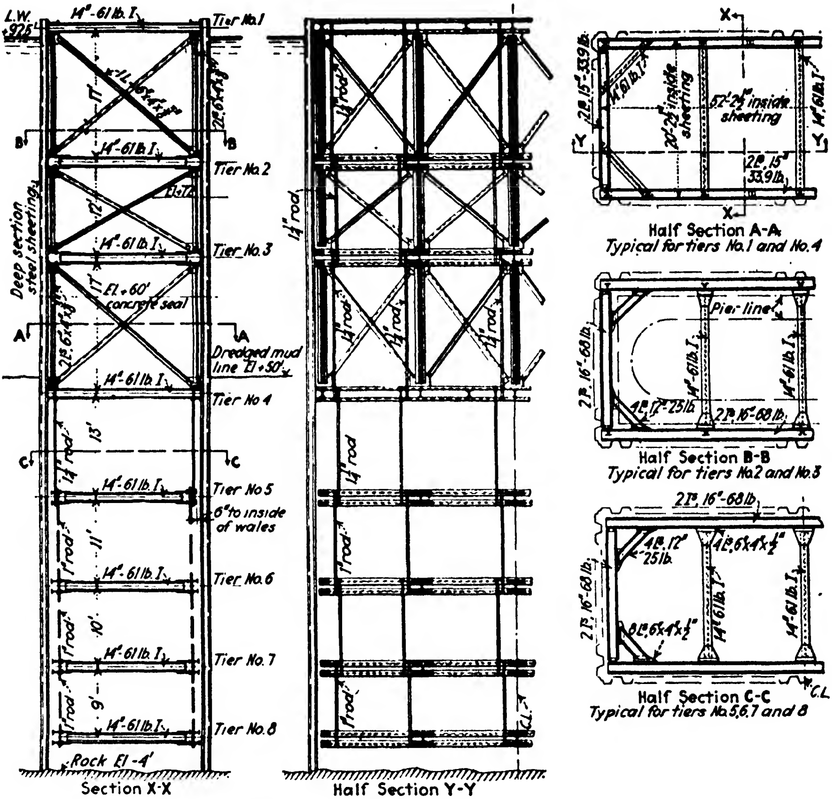


FIG. 8-5a.—Cofferdam Used in Placing Pier of the Lake Champlain Bridge at Crown Point State Park, N. Y.

to trim off this clay, which in some cases was as much as 2 ft. thick. Much of the jetted soil was pumped out with a centrifugal pump.

To permit sliding downward as excavation progressed, the four lower tiers of bracing were made 6 in. narrower in both directions than the dimensions of the cofferdam. As each tier reached its proper level, a diver inserted $1\frac{1}{4}$-in. rods connecting it to the tier above.

On completion of excavation the rock bottom was inspected by divers, after which concreting was done with 1-yd. bottom-dump buckets through water up to elevation +60. After this concrete had hardened, water was pumped out and the remainder of the concrete placed in the dry. From the top of seal to elevation +72 the pier shafts were 16 by 48 ft. in plan, and above this to elevation +92.5 they were 10 by 44 ft., the ends being semicircular.

When the work was completed, the sheet piling was pulled. The horizontal frames had to be left in below the elevation of sealing concrete, but above it all bracing was removed except the cross struts and diagonals passing through the concrete.

In building piers for the Jamestown bridge in Rhode Island, where the maximum depth of water was 70 ft., steel bracing cages for the cofferdams were assembled complete, transported to the site of the bridge, and lowered into the cofferdams by means of powerful floating derricks. The largest cage, 28 by 54 ft. in plan and 68 ft. high, weighed 158 tons. The sheet piling, which had a maximum length of 94 ft., was placed by means of timber guide frames supported on timber piles.

In constructing the 147- by 130-ft. cofferdam for one of the channel piers of the Meeker Avenue bridge over Newtown Creek in Brooklyn, N. Y., where the pier was founded on good bearing soil 48 ft. below mean high water, the site was first excavated and leveled off 22 ft. below water level. Temporary wooden piles were then driven both outside and inside the cofferdam, after which 3- by 10-in. batten plates were spiked to adjacent pairs of piles to form a platform on which to erect the cofferdam bracing cage. This bracing consisted of four tiers 35 ft. in height when in final position, with the upper tier 3 ft. above water level. The first three tiers were erected in a single unit in the form of a well-braced cage 25 ft. high. The fourth and lowest tier was temporarily supported just below the third tier by lashing it to the second tier.

After assembling the cage on the temporary platform, arched-web steel sheet piling, 64 ft. long, was driven to an elevation 34 ft. below water level, being guided on the outside by wales attached to the outer row of temporary timber piles and on the inside by the steel bracing.

The weight of the 160-ton bracing cage was then transferred to four 60-ton hydraulic jacks mounted on four elevated platforms inside the cofferdam carried by clusters of four piles. The cage was lowered with the same jacks until the third tier was in its proper position. The sheet piling was then driven home to an elevation

58 ft. below water level. The fourth and last tier was lowered about 10 ft. to its final position while the cofferdam was being excavated. This was done by the use of vertical threaded $1\frac{1}{8}$-in. rods 38 ft. long extending down from brackets on the first tier to connect with the fourth tier, nuts being turned to effect sinking.

After the completion of excavating work, the bottom was sealed with tremie concrete to an elevation 24 ft. below water level, the cofferdam was pumped out, and the remainder of the concrete placed in the dry.

8-6. Sheet Piling Supported by Cribs. For cofferdams which rest on hard bottom and are too large to employ internal bracing economically, a series of cribs, laid up log-house fashion, are used to hold the sheet piling in place. Each crib unit is made as long as can be conveniently handled and as wide as is necessary to develop the required stability. Rough logs are generally used, although in some cases they may be squared, but the latter offer only a slight advantage over the former. In building these cribs the bottom courses are usually started on land and the crib is built to a height sufficient to permit the top part being well out of water when it is first launched; after this it is launched, floated to place, and completed. Where the stream is low at certain times of the year, the cribs may sometimes be built in place. The bottom of each crib should be shaped to fit the rock bottom, and, if a few feet of sand or other material overlies the bedrock, this should be dredged out before placing the cribs. A part of the bottom of the crib is usually floored to permit placing stones so as to sink it.

After all the cribs are sunk, the remainder of the space inside of them may be filled with stones or earth. The latter material possesses the advantage of not only giving the cribs great stability but also securing watertightness. When the cribs are in place, sheet piling is driven around the outside and banked with earth. This type of cofferdam is very widely used in building dams for hydroelectric plants.

Figure 8-6*a* shows a view of the cofferdam employed in the construction of a dam for the Connecticut River Power Company, near Vernon, Vt. The width varied with the height of the cofferdam; for the upstream one the maximum width was 35 ft., and the maximum height was 42 ft., or 16 ft. above normal water level. The structure was of the rock-filled type made of round logs in 7-ft. checks, with the face logs slabbed on the sides to give good bearing for the sheet piling. The tops of the cribs were floored with logs to serve as a walk and also as a protection against ice pressures. On

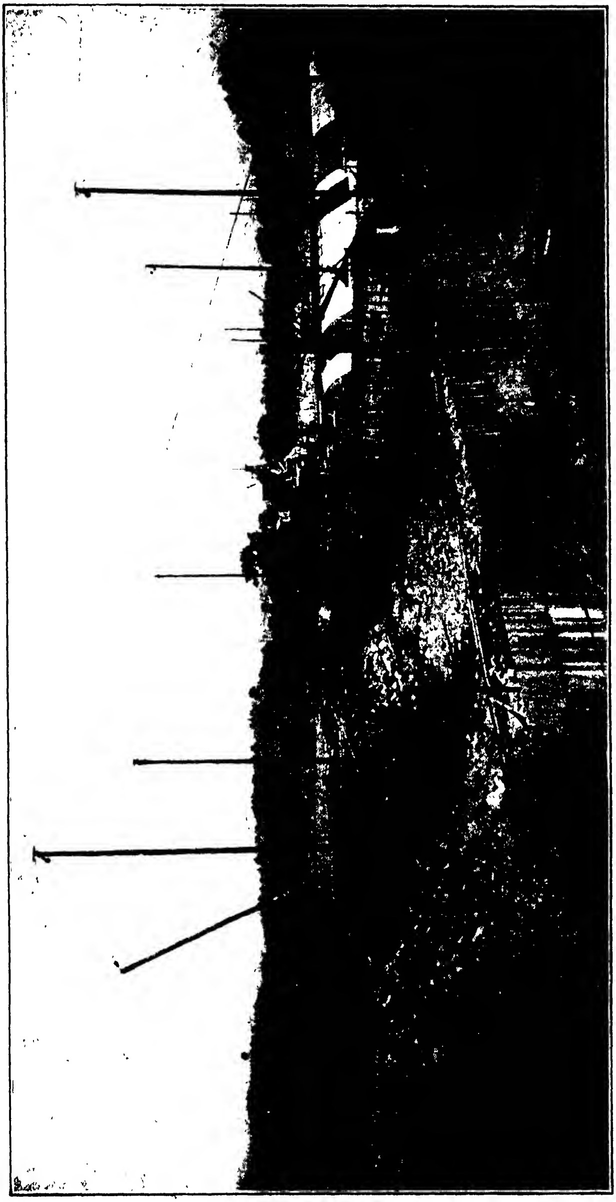


Fig. 8-6a.—Cofferdam of Connecticut River Power Company. Single Wall of Timber Sheet Piling Supported by Loaded Cribs.

the outside the cribs were sheet-piled with 3-in. spline-and-grooved spruce, and this in turn was banked with earth up to normal water level.

The cofferdams for the Niagara Power Plant of the Electrical Development Company of Ontario furnish an example of exceedingly strong and rigid cofferdams placed under the most trying conditions. In some places the current had a velocity as high as 17 ft. per sec., which made it difficult to study the nature of the bottom and the depth of water, previous to placing the cofferdams.

The widest part of the cofferdam consisted of two lines of parallel rock-filled timber cribs with a space between, sheet-piled and filled

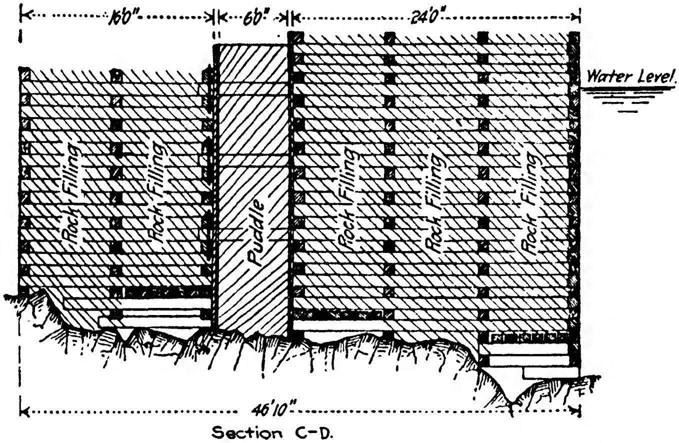


Fig. 8-6b.—Typical Section of Crib Cofferdam. Niagara Power Plant, Electrical Development Co. of Ontario.

with puddle as shown in Fig. 8-6b. Both cribs were built of squared timber with the outside wall of the outer crib laid solid. The width of the cribs varied to meet the variation in depth, and the bottom of the cribs was made to fit the irregularities of the rock surface. In shallow water the cribs were built in place, but elsewhere they were constructed in the river upstream, and by means of cables from the shore they were floated into place and were sunk by filling with rocks the wells which had bottoms.

8-7. Cellular Cofferdams. The cellular steel sheet-pile cofferdam is a type that has been used with success in unwatering large areas and consists of a series of connected cells filled with soil to furnish stability. One of two types is generally used, as shown in Fig. 8-7a. The left-hand type consists of a series of arcs connected to straight diaphragm walls. The radius of the arcs is usually made

equal to the distance between diaphragms, in which case the tensions in the arcs and cross walls are equal.

The other type consists of a series of complete circles connected by short arcs, the radius of the latter usually being about 8 ft. This type requires more material than the first type but has the advantage that each cell may be filled independently of other cells without distortion of the shell. With the first type the height of fill must be kept nearly uniform in all cells in order to avoid distortion in the cross walls.

A notable example of the diaphragm type of cellular cofferdam is that used in the construction of a ship pier at the foot of West

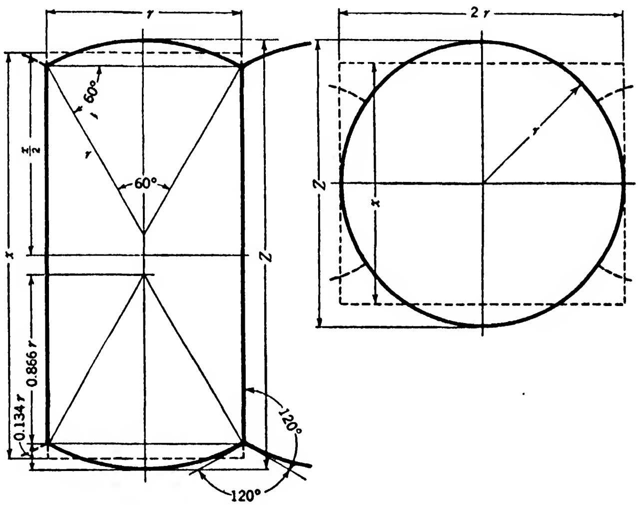


FIG. 8-7a.—Plan of Diaphragm and Circular-type Cells. (*Courtesy of Civil Engineering.*)

Forty-sixth Street, New York. It consisted of a core wall formed of cellular pockets of steel piling filled with earth, each pocket being about 16 ft. wide and 24 ft. long. The shore side of the wall was banked with riprap and the offshore side with earth, as shown in Fig. 8-7*b*. High water is 6 in. below the top of the cofferdam. The cells were made of Lackawanna piling weighing 37.2 lb. per ft., each cell consisting of two slightly rounded longitudinal walls with transverse connecting straight walls, the latter being braced with heavy channels bolted to each section of the piling. All the piling was in single pieces, many of which were over 70 ft. long.

The tension stress at the interlock of the piling may be analyzed as follows, it being a maximum when the cells are filled and no embankment around the outside. Assuming the weight of filling,

which was river mud, at 80 lb. per cu. ft. and its natural slope at $3\frac{1}{2}$ to 1, for a head of 58 ft., we have (see Art. 2-6)

$$p = wh \frac{(1 - \sin \phi)}{(1 + \sin \phi)} = 80 \times 58 \times 0.57 = 2{,}640,$$

where p denotes the pressure in pounds per square foot against the longitudinal walls at the bottom.

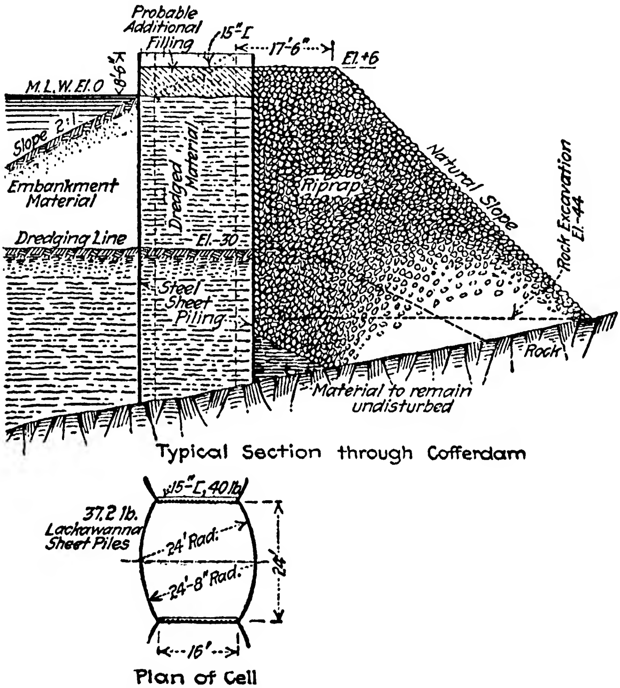


FIG. 8-7b.—Cellular Steel Sheet-pile Cofferdam.

If the filling is assumed to act radially,

$$t = \frac{pR}{12} = 2{,}640 \times \frac{24}{12} = 5{,}280,$$

where t denotes the tension in pounds per linear inch in the piling of the longitudinal walls and R the radius of curvature in feet. Likewise

$$t' = \frac{pd}{12} = 2{,}640 \times \frac{24}{12} = 5{,}280,$$

where t' denotes the tension in pounds per linear inch in the piling of the transverse walls and d the spacing of the transverse walls in

feet. Tests made on the piling previous to driving showed a strength at the interlock of 9,000 lb. per lin. in.

One of the first examples of the diaphragm type of cellular cofferdam was the one used at Black Rock Harbor, Buffalo, built to permit the construction of a ship lock, as shown in Fig. 8-7*c*. The depth of water at the site varied from 2 to 15 ft., averaging about 8 ft.; the solid rock on which the lock was built was about 40 ft. below mean water level. As shown in Fig. 8-7*d*, the sides of the cofferdam were made of two walls of steel sheet piling, the space between the two walls being divided into pockets 30 ft. square by transverse walls of the same type of piling as that used for the main walls, which served to connect the latter. Horizontal 15-in. 40-lb. channels were bolted

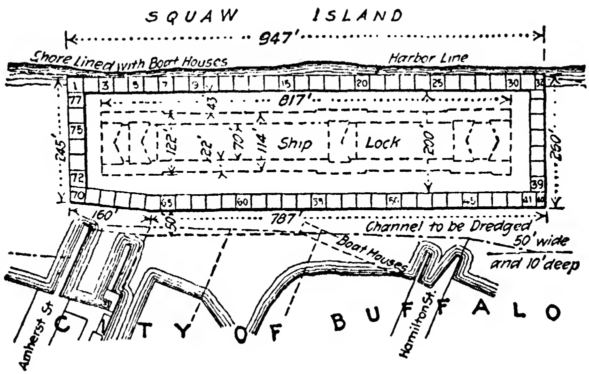


FIG. 8-7*c*.—Plan of Black Rock Cofferdam.

to the tops of the piles of the inner wall, and similar channels were bolted at an inclination across the transverse walls, as shown in Fig. 8-7*c*.

The piling was driven to rock, and at first wood guide piles and wales were used to maintain the alignment of the steel sheeting. But eventually these guides were dispensed with, the only ones used being 10- by 30-ft. floating forms having one edge in the plane of the sheeting. The fine alignment attained by this simple method may be seen in Fig. 8-7*d*. After driving the piling, the pockets were filled with clay, and, to further strengthen the structure as the inside was excavated, a bank of earth 25 ft. high was maintained on the inside, as shown in Fig. 8-7*e*. But in spite of the bank of earth the material in the pockets caused the inside wall to bulge badly between the cross walls in both a horizontal and vertical direction. This demonstrated that curved longitudinal walls made the better design and the last four pockets were constructed in this way.

Fig. 8-7*d*.—Cofferdam for the U. S. Government Ship Lock at Black Harbor. Northeast Corner.

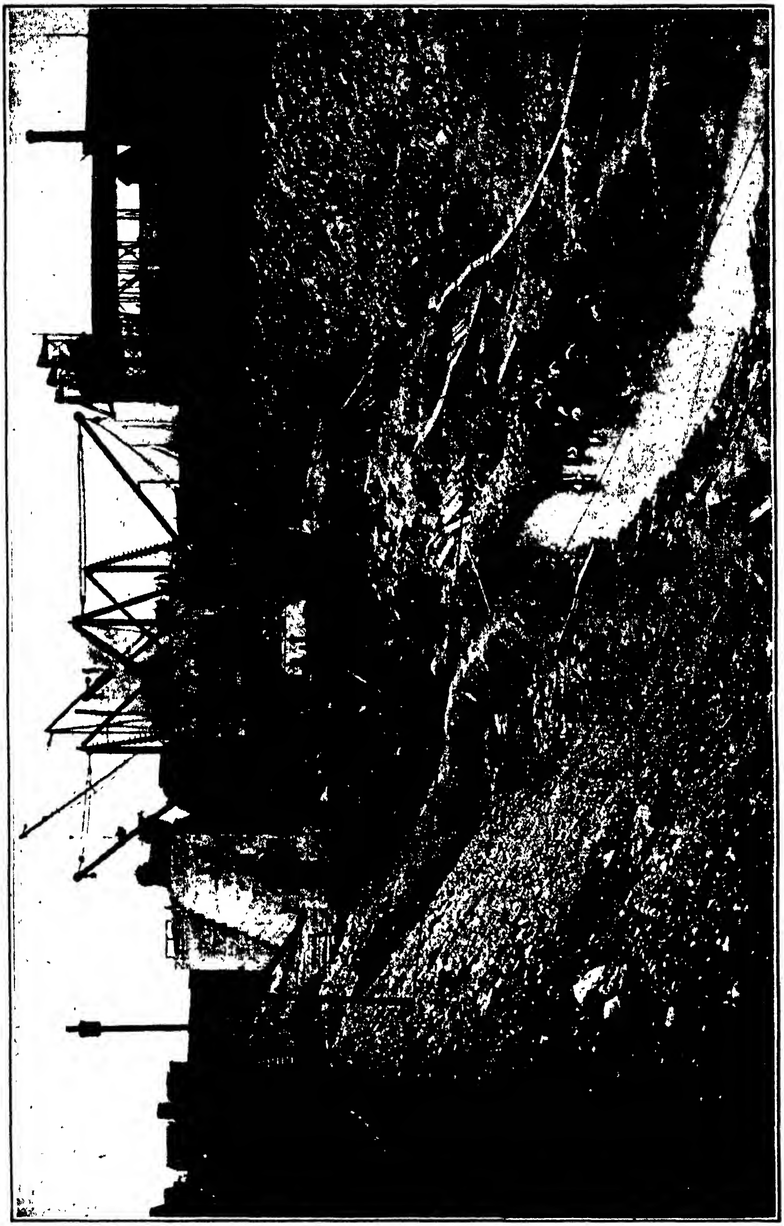


FIG. 8-7e.—Cofferdam at Black Harbor, Looking North from Southwest Corner. Shows Bulging of Pockets.

The cofferdams, used in building the New Jersey tower piers of the George Washington bridge across the Hudson River at New York in 1927, had straight-wall cells for the front and for the two sides where the water was deep and a single wall on the back and on a part of the sides where the water was not so deep. These

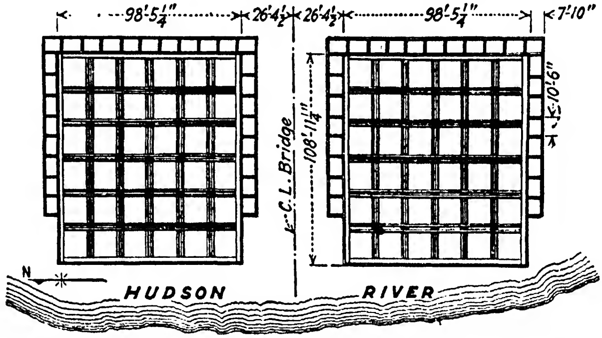


FIG. 8-7*f*.—Plan of Twin Cofferdams Used in Building New Jersey Tower Piers of the George Washington Bridge.

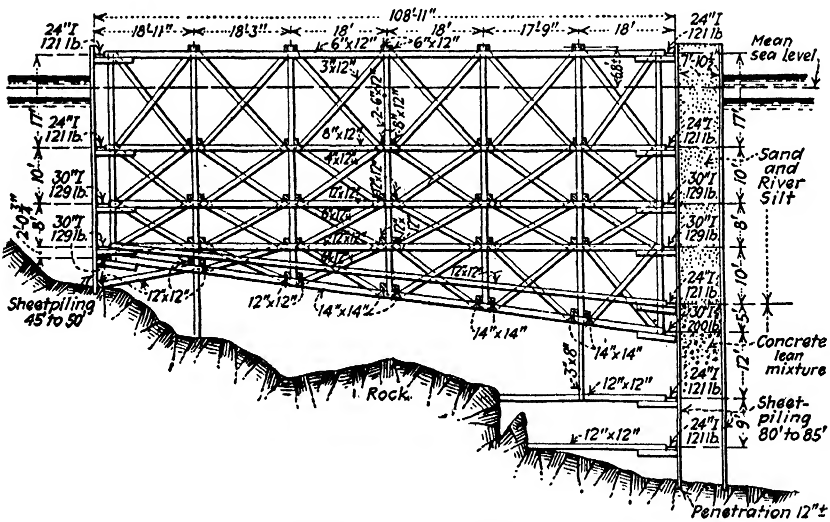


FIG. 8-7*g*.—Longitudinal Section of North Cofferdam.

structures differed from most cellular cofferdams in that heavy internal bracing was used.

As shown in Fig. 8-7*f*, each cofferdam was approximately 98½ by 109 ft. in plan. The maximum head of water was 79 ft. Bedrock sloped steeply toward the river, its depth below mean tide varying from 32 to 38 ft. at the inshore ends and from 65 to 79 ft. at the

outshore ends. A stratum of shells and sand 4 to 5 ft. thick covered the rock; above this was Hudson River silt, which ranged in consistency from stiff clay at the bottom to almost fluid mud at the top.

The entire area was first dredged to rock inshore and to an elevation 45 ft. below mean sea level at the outer edges of the cofferdams, the original depth of water having been about 20 ft. The bottom set of frames for the cofferdam bracing had meantime been built. This was floated into place, guide piles driven, and the remaining tiers built up in place. As shown in Fig. 8-7g, these frames consisted of I-beam walers as well as timber cross bracing made up of five struts running north and south and five struts running east and west. The walers varied from 24-in. 121-lb. to 30-in. 200-lb. I-beams, and the struts varied from two 6- by 12-in. timbers in the top set to two 14- by 14-in. timbers in the bottom set. All timbers were securely bolted together, with vertical posts between the different tiers and also cross bracing tying the struts together into solid truss construction.

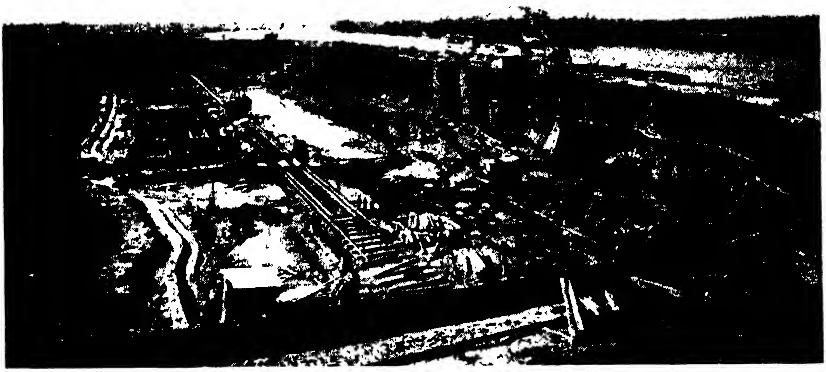


Fig. 8-7h.—Lock Cofferdam at Pickwick Landing Dam.

After the frames were completed and lowered to the desired position, the sheet piling was driven around them. The pockets were dredged out and backfilled with concrete part way up and with sand and silt above. After unwatering, excavation was completed, additional bracing was placed where needed, and the piers built.

Figure 8-7h shows the circular-cell sheet-pile cofferdam used in constructing the lock of the Pickwick Landing Dam of the Tennessee Valley Authority.[1] Each cell was 58.89 ft. in diameter and

[1] *Civil Eng.*, vol. 9, p. 551, September, 1939.

was formed of 15-in. piling with a $\frac{3}{8}$-in. web. The cells were filled with sand and gravel, and embankment material was placed along the outside. This cofferdam had a head of 55 ft., measured from the top of the cofferdam to the base of the sheet piling, which rested directly on solid rock.

There was a total of 13,800 tons of piling driven and pulled on this job, the costs being as follows:

	Cost Per Ton
Handling and hauling	$1.405
Setting piling	5.602
Templets	2.541
Driving	3.639
Pile pulling	6.279

The first use of the circular-cell type of cofferdam was in raising the *Maine* in Havana Harbor, as illustrated in Fig. 8-7*i*. The piles

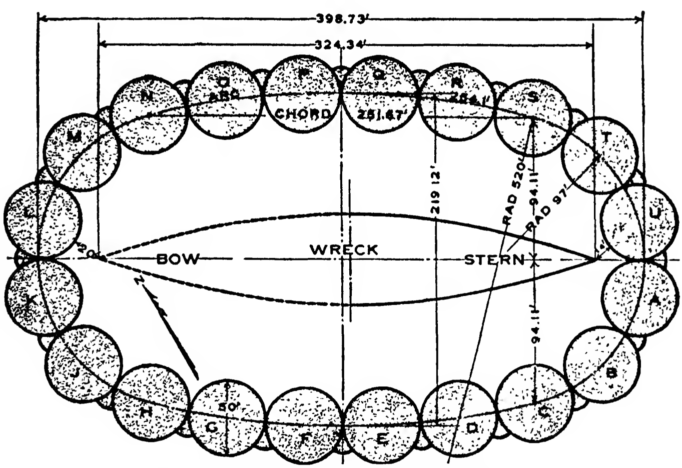


FIG. 8-7*i*.—Plan of Cofferdam for Raising the *Maine*.

were driven so that their tops were 2 or 3 ft. above normal water level (Fig. 8-7*j*), and the 75-ft. length of piling, which penetrated the harbor bottom to a distance of approximately 35 ft., was made in two lengths spliced together with channels. It was planned to fill the cylinders with heavy clay, the thought being that this clay would displace the 25 ft. of harbor silt in the cylinders. As time did not permit the use of dipper dredges for this work, hydraulic dredges were employed, and as a result the new material blanketed the existing harbor silt in the cylinders. When the cofferdam was

ready to be unwatered, it was found impossible to solidify this silt, and it remained in a semifluid condition throughout the work.

As the water was pumped out inside the cofferdam, the cylinders became badly distorted, taking an elliptical form and bulging inward. To prevent failure, riprap was placed against the inside of the cofferdam and the filling of the cylinders was shifted somewhat to form approximately a continuation of the riprap slope. Before the cofferdam could be completely unwatered, a number of

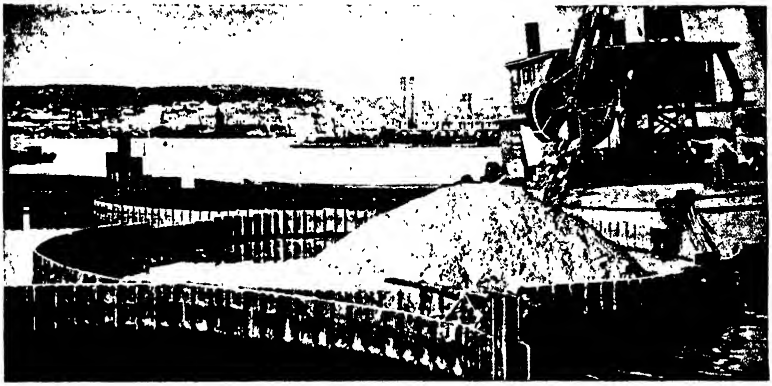


FIG. 8-7*j*.—Filling Clay into Cylinder *A*. Part of *B* in Foreground.

heavy braces had to be set between the cylinders and the wreckage of the *Maine*.

8-8. Movable Cofferdams. Where the same size and style of cofferdam are to be used for a number of piers, it will often prove advantageous to construct cofferdams that can be used over and over again. This is particularly true in the case of cofferdams on grillage.

Figure 8-8*a* illustrates the type of movable cofferdam used in constructing the piers of the Key West Extension of the Florida East Coast Railway, where the depth of water did not exceed 8 ft. The two sides and the two ends formed independent portable sections connected together with $1\frac{1}{4}$-in. vertical rods running down through the overlapping rangers at the corners of the cofferdam. At the site of the piers sand overlaid the coral rock. Piles for the foundation of a pier were driven until the tops were 2 ft. below low water. The cofferdam was assembled on a barge, lifted from the same, and set in place. The sand was then pumped out to rock by a centrifugal pump, and a 2-ft. seal of concrete was placed through the water. After allowing this concrete to harden for 7 days, the cofferdam was pumped out, forms placed, and the pier

built. On completion of the pier the rods were withdrawn to allow the sections to float free for use again.

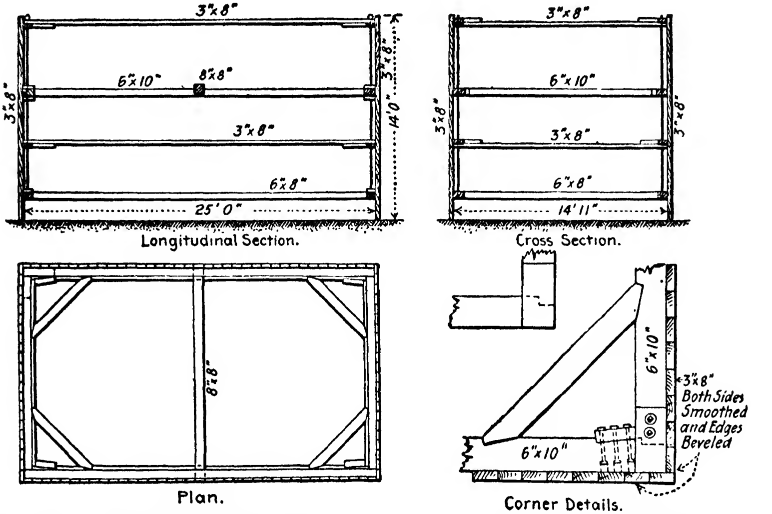


FIG. 8-8a.—Cofferdam Used on Key West Extension of Florida East Coast Railway.

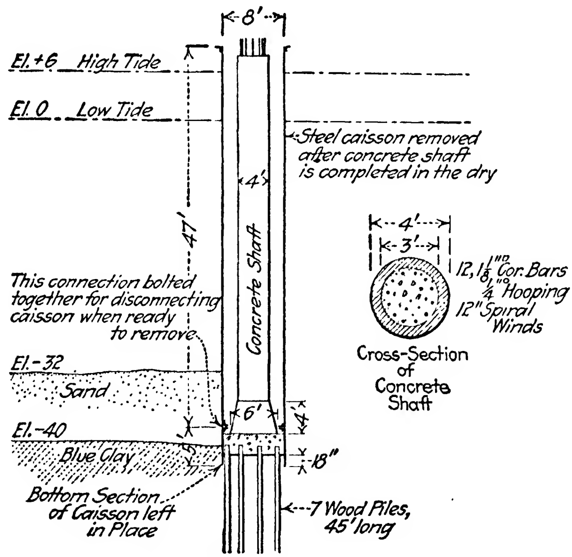


FIG. 8-8b.—Removable Cylinder Cofferdam.

A novel type of movable cofferdam, shown in Fig. 8-8b, was used in Charleston, S. C., in which to build 4-ft. concrete cylinder shafts. An 8-ft. steel cylinder of ⅜-in. plate, the lower 5 ft. of which

was attached to the part above by a butt joint, was driven into the river bed and the inside excavated with an orange-peel bucket to a

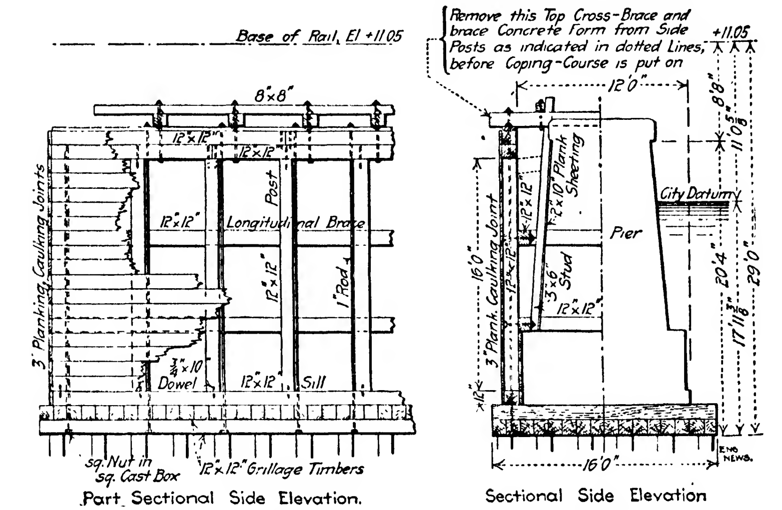


FIG. 8-8c.—Cofferdam for Rest Pier at Chicago and Northwestern Railway Lift Bridge at Kinzie Street, Chicago.

short distance above the cutting edge. The water was pumped out and the bottom leveled by hand-digging. Foundation piles were then driven, in some cases the piles being driven in the dry and in other cases through the water. However, before cutting off the piles, the water was pumped out. After the piles were cut off, a 3-ft. bottom course of concrete was placed in the dry. The concrete shaft was built up in forms, and the cylinder, except for the lower 5-ft. section, was lifted and moved to another location.

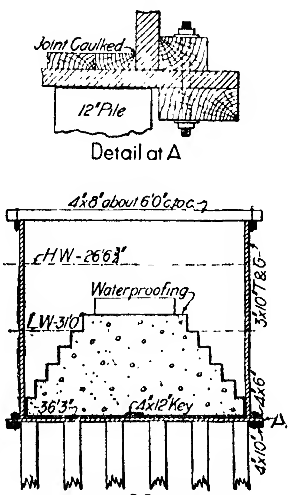


FIG. 8-8d.—Cofferdam with Removable Sides.

Another type of movable cofferdam is that in which the structure is built with a floor which remains as a permanent part of the pier. This floor, which may be of timber or of concrete, forms, with the sides, an open box. The cofferdam, which is usually built on shore or on a barge or raft, is floated to the site and sunk on a pile foundation by building the pier in the box. This type differs from the

box caisson described in Art. 9-2, since the sides of the cofferdam are not a permanent part of the pier. After the pier is built to above high-water level, the cofferdam sides are removed.

Figure 8-8c shows the details of the movable cofferdam on timber grillage used for the 12- by 41½-ft. pier of the Kinzie Street drawbridge in Chicago. This cofferdam was connected to the grillage by 28 vertical 1-in. rods 21½ ft. long. To sink the structure, con-

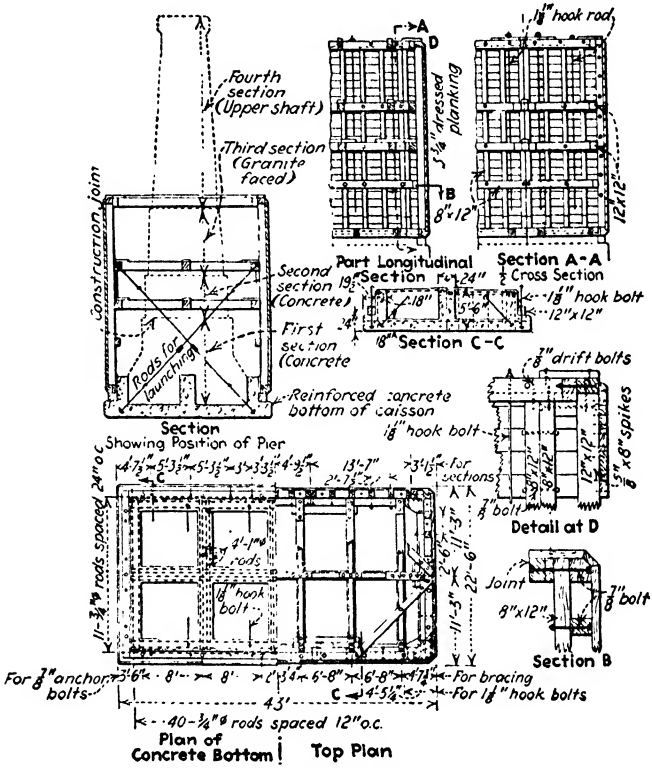


FIG. 8-8e.—Movable Cofferdam with Concrete Base for Newark Bay Bridge of the Central Railroad of New Jersey.

crete forming the pier was placed, the cofferdam itself serving as a form for the concrete for a certain height, and above this, regular forms were used. When the pier was completed, the rods were removed, thus permitting the separation of the cofferdam from the grillage.

A very simple cofferdam on grillage was used in building the foundation piers of the Bellevue Hospital (New York) boilerhouse, a section of which may be seen in Fig. 8-8*d*. The most interesting feature is the very thin grillage used, it being composed of two

crossed courses of 2-in. tongue-and-grooved planks. It was desired to use a thickness which would provide enough strength to resist the launching and sinking stresses and yet be sufficiently flexible to secure a uniform bearing on the slightly irregular pile tops.

The cofferdams for the Newark Bay bridge of the Central Railroad of New Jersey had a reinforced-concrete base, as shown in Fig. 8-8*e*.

The reinforced-concrete bottom was cast on a cradle on launching ways near the work and was prepared for launching after seasoning for 21 days. The six side sections and two ends were then set in place and thoroughly caulked along the base and their vertical joints, and the completed cofferdam was then launched. At the site of the pier to be constructed, piles had been driven and cut off by this time, and the cofferdam was sunk to place on the piles. The pier was built inside the cofferdam and was usually carried to completion before the sides were removed. The latter operation required merely removing the bolts between the sections, turning the long hook bolts passing down through the sides which held the side sections to the base, and floating the sections away separately.[1]

The bottom thickness as designed was 18 in., which later was increased to 24 in. because the thinner section developed some leaks.

For another example of a large cofferdam of this type see an article entitled Floating-caisson Construction of Bascule Bridge Piers, *Engineering News-Record*, vol. 99, p. 48, July 14, 1927.

8-9. Puddle and Leakage. The construction of a watertight cofferdam is seldom attempted, for the cost of such is usually prohibitive. The greatest trouble from leakage occurs in the case of cofferdams which rest on rock, for here it is almost impossible to prevent the water from running in between the bottom of the cofferdam and the rock. But, if the structure rests on clay and the sheet piling is driven well down, there will be but a slight amount of leakage from below. Where the leakage occurs through seams in the rock, it may be stopped by filling the seams with grout pumped in through pipes about 4 in. in diameter. Another method is to dump clay, sand, ashes, etc., all around the cofferdam with a view of shutting off the water supply to the crevices.

When the leakage is due to irregularities in the rock surface, concrete in bags may be placed on the bottom, or water-logged oat straw may be sunk by mixing it with ashes or covering it with a wire net loaded with sand and clay, after which the rest of the puddle filling may be placed. Another method of preventing leakage on a rock bottom is to use canvas as noted in a previous article.

[1] *Eng. News-Record*, vol. 96, p. 184, Feb. 4, 1926.

Leaks often develop in double-wall cofferdams by the filling between the walls not compacting well, or settling after being placed and leaving openings beneath cross braces. To compact this filling, piles are sometimes driven into it or stock ramming may be resorted to. The latter consists of forcing clay cylinders through pipes into the filling. Another method sometimes used is to drive a hole in the puddle above the leak and to ram in quantities of excelsior. This excelsior swells quickly when wet and also acts as a filter.

With single-wall steel sheet piling the joints above ground level often leak excessively. This may be prevented by putting down outside of each joint a V-shaped wood trough consisting of two boards nailed edge to edge and filling the space between with puddle. In one case leakage was stopped by running a box filled with a mixture of sawdust and ashes along the joints at the points of leakage. The box was handled by a long pole nailed to the same, and the open top of the box was placed against the joint. The inward flow of the water drew the contents of the box into the joint very effectively.

In building the piers of a new highway bridge across the Arkansas River, the cofferdams were constructed of a single wall of steel sheet piling. The steel sheeting was driven with extreme difficulty to what was thought to be bedrock, but an attempt to unwater the inclosure showed that the difficulty in driving was due to a massed boulder formation overlying sound rock. The formation was a mixture of sand, coarse gravel, and boulders. To remedy the situation, a line of 2-in. pipes at 3-ft. intervals was driven about 1 ft. from the outside of the sheeting. The pipes averaged about 35 ft. in length. After driving to refusal, the pipes were cleaned out with a jet and chopping bit.

Grout was then mixed in an open 4- by 12-ft. tank, using about 5 gal. of water to a sack of cement. A quantity of grout containing three sacks of cement was pumped into each pipe. It required a pressure of about 100 lb. to start the grout through the pipe, after which it required only about half that pressure. When a low pressure started the grout through the pipe, the quantity used was increased, and in some cases as many as eight sacks were pumped into one pipe. Near the close of the pumping operation the pipe was slowly lifted from the rock. Each cofferdam required about 300 sacks of cement for a complete seal.

The best puddle for a cofferdam is a mixture of clay and sand or clay and gravel. The clay should be cohesive and impermeable to water. The cohesive quality may be tested by working it with a small quantity of water and then forming it into a cylinder $1\frac{1}{2}$ in.

in diameter and 10 in. long. If, when suspended at one end while wet, it does not break, it has sufficient cohesion. To test the degree of impermeability, a considerable quantity should be worked into a plastic mass, hollowed out, and filled with water. The clay should hold this water for some time.

In making puddle, only enough sand should be added to the clay to prevent cracking by shrinkage in drying. Puddle should be placed in layers about 3 in. thick and should be well chopped with spades, water being added at the same time. The spade should pass through two layers to ensure bonding the layers together.

8-10. Design of Cofferdams. Like most structures used in foundations, a purely theoretical design of cofferdams leads to unsatisfactory results. For some types it is a simple matter to design the structure to resist the hydrostatic pressure, but to design it properly to resist safely the pressure of the earth filling, or of freshets, ice, or floating logs requires much experience.

Earth cofferdams usually fail by the water seeping through and enlarging a channel until a washout takes place. For this reason such cofferdams should be carefully watched to detect small leaks that they may be checked quickly after starting. In general, if the cofferdam is made of a good mixture of clay and sand, has a width of at least 3 ft. at the top, which is well above high water, and has sides inclined at the natural slope of the material, the cofferdam will be safe.

In the single-wall sheet-pile cofferdam with guide piles, if the wales are at the top and bottom, the sheet piles may be assumed to act as simple beams, with a load per vertical foot varying uniformly from zero at the water surface to wd lb. per sq. ft. at the surface of the earth, where w is the weight in pounds of 1 cu. ft. of water and d the depth of water in feet. The reaction at the upper wale is $wd^2/6$ and at the lower wale $wd^2/3$, both in pounds per linear foot of wall. The maximum bending moment in the sheet piling occurs at a point $0.577d$ down from the top and equals $0.064wd^3$ ft.-lb. per ft. of wall. The wales take the reactions of the sheet piling and transfer them to the guide piles as supported, partially continuous or continuous beams. Conservative engineers usually design the wales as simple or supported beams. If no bracing is used, the guide piles should be designed as cantilever beams (Art. 7-17).

Each wall of the double-wall sheet-pile cofferdam with guide piles may be designed somewhat in accordance with the above outline. The outer wall will be subjected to water pressure from the

outside and earth pressure from the inside. Experience shows that usually the pressure from the puddle will, for equal heads, be larger than the pressure from the water. This will cause a stress in the tie rods connecting the two walls. The inner wall must be designed to resist the forces due to the puddle filling.

In the design of a cofferdam composed of sheet piling on frames and the corresponding bracing, the outside pressure is the only force to be considered. The sheet piling acts as a beam between horizontal rangers, and the latter act as beams between bracing struts. Article 8-11 contains an example of this type of design.

The sheet-pile-on-crib cofferdam must be designed so that the cribs will not overturn or slide. To be safe against sliding, the weight of the cribs and the filling per linear foot of length, multiplied by the coefficient of friction between the crib and rock, must be greater than $wd^2/2$, where the terms have the same meaning as those previously given. To be safe against overturning, the weight per linear foot of length, including filling, multiplied by one-half the width, must be greater than $wd^3/6$. To be safe against uplift at the heel, the weight must be multiplied by one-sixth of the width instead of one-half.

The design of cellular steel sheet-pile cofferdams is treated in Art. 8-12.

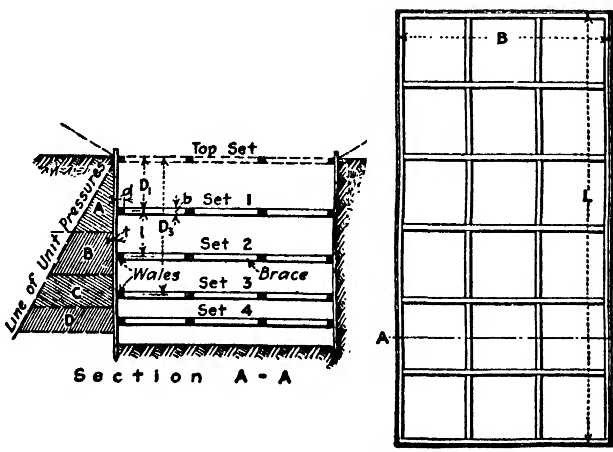


FIG. 8-11*a*.—Plan, Section, and Outside Pressure Diagram of Single-wall Cofferdam.

8-11. Design of Single-wall Cofferdams. To get the vertical spacing of waling pieces of equal strength for the type of cofferdam shown in Fig. 8-11*a*, the following formula[1] may be used:

[1] See *Eng. News-Record*, vol. 82, p. 708, Apr. 10, 1919.

$$D_n = 0.314 \frac{k^{\frac{1}{2}}b^{\frac{1}{2}}d}{p^{\frac{1}{2}}s} [N^{\frac{3}{2}} - (N - 1)^{\frac{3}{2}}] \cdot \cdot \cdot \qquad (8\text{-}11a)$$

where p denotes the lateral fluid or earth pressure in pounds per square foot one foot below the surface, s the span of the wales in feet, k the allowable unit stress in pounds per square inch of timber in bending, b the width in inches of the wales acting as beams, d the depth of same, D the distance in feet from the surface to the wale in question, and N the number of rows of wales from the surface not counting the top row.

This formula is developed on the basis that the top waling takes no load, the load from the areas A, B, C, etc., going, respectively, to the waling sets 1, 2, 3, etc. Since the wales are of equal strength, these areas will be equal. If W denotes the allowable load on each wale acting as a simple beam and h the total depth in feet,

$$\frac{ph^2s}{2} = NW = \frac{Nkbd^2}{9s} \cdot \cdot \cdot \qquad (1)$$

If we take moments about the top and equate the moment of the external pressure to the moment of the loads on the wales,

$$\frac{2NWh}{3} = W(D_1 + D_2 + D_3 \cdot \cdot \cdot + D_{n-1} + D_n) \qquad (2)$$

$$D_n = \frac{2}{3}\sqrt{\frac{kbd^2N^3}{4.5ps^2}} - (D_1 + D_2 + D_3 \cdot \cdot \cdot + D_{n-1}) \qquad (3)$$

$$D_{n-1} = \frac{2}{3}\sqrt{\frac{kbd^2(N-1)^3}{4.5ps^2}} - (D_1 + D_2 + D_3 \cdot \cdot \cdot + D_{n-2}). \qquad (4)$$

Equation 8-11a is obtained by substituting Eq. (4) in Eq. (3).

The values of $[N^{\frac{3}{2}} - (N - 1)^{\frac{3}{2}}]$ for various values of N are as follows,

N	1	2	3	4	5
$[N^{\frac{3}{2}} - (N - 1)^{\frac{3}{2}}]$	1.00	1.83	2.37	2.80	3.18
N	6	7	8	9	10
$[N^{\frac{3}{2}} - (N - 1)^{\frac{3}{2}}]$	3.52	3.82	4.11	4.37	4.62

Assuming that the sheet piling acts as a simple beam between wales, including the top wale, the highest stressed section will be between set 1 and set 2, and the bending moment may be determined without serious error on the assumption that the load is uniformly distributed.

For hydrostatic conditions and an allowable unit stress in the timber of 1,500 lb. per sq. in., Eq. (8-11a) reduces to

$$D_n = 1.538 \frac{b^{\frac{1}{2}}d}{s} [N^{\frac{3}{2}} - (N - 1)^{\frac{3}{2}}], \qquad (8\text{-}11b)$$

and with earth-pressure conditions corresponding to a 30-lb. equivalent fluid and the same allowable unit stress in the timber as above it becomes

$$D_n = 2.22 \frac{b^{\frac{1}{2}}d}{s} [N^{\frac{3}{2}} - (N - 1)^{\frac{3}{2}}]. \qquad (8\text{-}11c)$$

The load on each strut is given by the formula $kbd^2/9s$.

EXAMPLE. A cofferdam 30 ft. deep is to be designed for hydrostatic conditions, with 12- by 12-in. wales, an allowable unit fiber stress in bending of 1,500 lb. per sq. in. and a strut spacing of 8 ft. By Eq. (8-11*b*) the values of D_n are 8.0, 14.6, 19.0, 22.4, 25.4, and 28.2 ft.

The maximum moment in the sheet piling is

$$\frac{62.5 \times 11.3 \times 6.6^2 \times 12}{8} = 46{,}100 \text{ in.-lb.}$$

If timber piling is used,

$$\frac{1{,}500 \times 12t^2}{6} = 46{,}100 \quad t = 3.92 \text{ in.}$$

hence 4-in. piling will be used. If steel piling is used, the required section modulus per horizontal foot of wall will be

$$\frac{46{,}100}{18{,}000} = 2.56 \text{ in.}^3$$

from which a section may be picked out of a structural handbook.

The load on each strut is

$$\frac{1{,}500 \times 12^3}{9 \times 8} = 36{,}000 \text{ lb.}$$

If a minimum diameter of 6 in. is assumed, the unit fiber stress allowed by the formula $1{,}500\left(1 - \frac{l}{60d}\right)$ is 1,080 lb. per sq. in. If we divide this into 36,000, the required area of strut is found to be 33.3 sq. in.; therefore a 6- by 6-in. strut will be used.

8-12. Design of Cellular Cofferdams. A cellular cofferdam is essentially a gravity-type bulkhead and is designed as such (Art. 7-19). However, owing to the difference in shape, the design formulas will differ somewhat. The external lateral loads and the position of their resultant will be found as in Art. 7-19. The width of the cofferdam will be determined on the principle that there shall be no tension at the heel, in which case the unit uniform compression due to the vertical loads will equal the maximum unit tension due to the lateral forces, or

$$\frac{W}{A} = \frac{Mc}{I} = \frac{rPy}{I}\frac{Z}{2},$$

where W = weight of fill in one cell in pounds
A = area in square feet of plan of one cell
P = resultant lateral force in pounds for 1 ft. length of cofferdam
y = distance above base in feet of P
Z = maximum width in feet of cell
I = moment of inertia in feet^4 of area of plan of one cell

For the type illustrated in Fig. 8-7a (left-hand figure) this formula becomes approximately,

$$\frac{(w_e h' + w_{e \text{ in } w} h'')A}{A} = \frac{rPy\dfrac{Z}{2}}{\dfrac{1}{12}rZ^3 - 4 \times \dfrac{1}{2} \times \dfrac{r}{2} \times 0.134r\left(\dfrac{Z}{2} - \dfrac{1}{3} \times 0.134r\right)^2},$$

where r is the width of cell and radius of outer walls in feet, and other terms as in Art. 7-19. This reduces to the following:

For $\dfrac{Z}{r} = 2$

$$Z = \sqrt{\frac{7.3Py}{w_e h' + w_{e \text{ in } w} h''}}.$$

For $\dfrac{Z}{r} = 3$

$$Z = \sqrt{\frac{6.9Py}{w_e h' + w_{e \text{ in } w} h''}}.$$

For $\dfrac{Z}{r} = 4$

$$Z = \sqrt{\frac{6.6Py}{w_e h' + w_{e \text{ in } w} h''}}.$$

For the circular cofferdam, Fig. 8-7a (right-hand figure), we have,

$$\frac{(w_e h' + w_{e \text{ in } w} h'')A}{A} = \frac{ZPy\dfrac{Z}{2}}{\dfrac{1}{4}\pi\left(\dfrac{Z}{2}\right)^4},$$

or

$$Z = \sqrt{\frac{10.2Py}{w_e h' + w_{e \text{ in } w} h''}}.$$

The critical stress in the sheet piling is that of tension in the interlock at ground level. For the circular cofferdam and also for the diaphragm-wall type where the radius of the curve of the outside walls equals the distance between the walls, we have

$$t = \frac{pr}{12}$$

where t = tension in the interlock in pounds per lineal inch
p = lateral pressure from the fill in pounds per square foot
r = radius of the curve in feet

The safety against sliding and internal shear is determined as outlined in Art. 7-19. The maximum intensity of internal shear for the circular cofferdam is $0.85 \frac{P}{r}$, whereas for the diaphragm type it may be taken as $\frac{3}{2}Pr/A$, where A is the area in square feet of a cell This will give a value somewhat high but on the safe side.

CHAPTER IX

BOX AND OPEN CAISSONS

9-1. Definitions and Classification. In this book caissons for bridges and buildings are divided into three general types: box caissons, open caissons, and pneumatic caissons. These types may be further subdivided according to the material used, namely, timber, metal, or concrete. All caissons have one characteristic in common; they form a permanent shell for, and are an integral part of, bridge and building foundations, being used simply as a convenient means of placing the foundation in position.

The box caisson is used where no excavating is required, and consists merely of a box, open at the top and closed at the bottom, which is filled with concrete or stonemasonry to serve as a foundation for the pier or other structure to be built on the same. Where sinking through soil must be resorted to in order to carry the foundation down to a stratum having sufficient bearing power to carry the superincumbent load, the box must be open at the bottom in order that the earth underneath may be removed. If this excavating is done through the water, the structure is called an "open caisson"; if the caisson is roofed and air pressure used to force out the water below in order that workmen may enter to remove the material by hand, the structure is called a "pneumatic caisson."

Briefly then, a caisson is a box; a box caisson if open at the top and closed at the bottom, an open caisson if open at the top and bottom, and a pneumatic caisson if closed at the top and open at the bottom and uses compressed air. In all cases the caisson is merely a shell, which must be filled with concrete or other masonry. Most caissons are surmounted with cofferdams because, from standpoints of appearance and durability, it is usually undesirable to extend the caisson above low-water level.

Open caissons may be classified as (*a*) single-wall open caissons, (*b*) cylindrical open caissons, and (*c*) open caissons with dredging wells.

The first caissons in this country placed by open dredging were those sunk in 1883 for a railroad bridge across the Atchafalaya River at Melville, La. They were metal cylinders 8 ft. in diameter

and were sunk to a maximum depth of 120 ft. below high water, or from 70 to 115 ft. below mud line. The clamshell type of dredge was first used on this work.

The second application of the open-caisson method was in 1888 for the Poughkeepsie bridge over the Hudson River. The largest caisson of this bridge was 60 by 100 ft. in plan, and it rested on a bed of gravel 134 ft. below high water. In some details, such as filling the pockets with gravel and using a removable cofferdam on grillage, the construction of these caissons differed materially from what is now standard practice.

The largest and deepest caissons ever built are those used in the San Francisco-Oakland bridge completed in 1937. The largest caisson of this bridge is 92 by 197 ft. in plan. The maximum distance from water surface to bottom of the foundation is 220 ft. Another caisson for this bridge was sunk to a depth of 242 ft.

Among the notable foundations placed by the open-caisson method are the following, these being outstanding either because of their depth or because of their size, or both. The maximum depth was not always with the maximum size.

Date	Bridge	Dimensions in feet, largest caisson	Maximum depth, feet	Material
1936	San Francisco-Oakland	92 × 197	242	Steel and concrete
1939	Tacoma Narrows	66 × 120	224	Steel and concrete
1938	Baton Rouge	63 × 82	180	Concrete
1935	New Orleans	65 × 102	170	Concrete
1933	Atchafalaya River at Morgan City, La.	44 ft. diameter	176½	Steel
1886	Hawkesbury, Australia	20 × 48	162	Steel
1915	Hardinge over lower Ganges	35 × 63	160	Steel
1928	Mid-Hudson	60 × 136	135	Steel and concrete
1888	Poughkeepsie	60 × 100	134	Timber

9-2. Box Caissons. Box caissons are made of timber and of concrete, the former material being more widely employed than the latter. Except where placed on piles, the use of this type of caisson is limited, owing to the necessity of first excavating to the desired depth, that is, to where firm bearing may be obtained, before placing

the caisson. The depth to which it is possible to excavate is limited, owing to the tendency of the wet material to flow into the hole. In a few cases box caissons have been made to sink several feet by running pipes through the bottom and forcing water through the

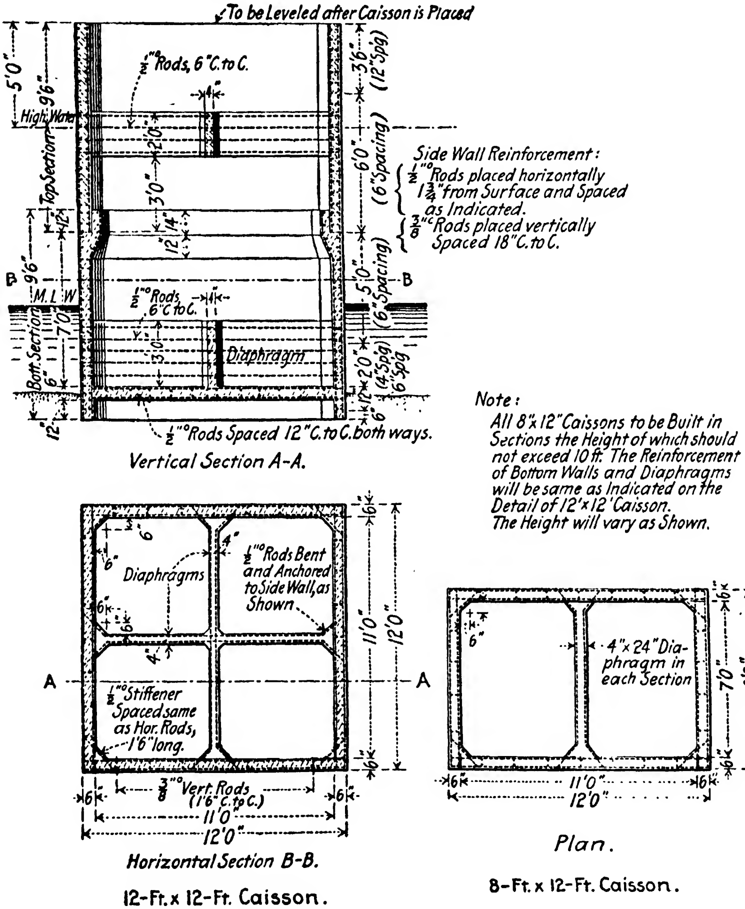


FIG. 9-2a.—Box Caisson of Reinforced Concrete near Glen Cove, Long Island.

same, thus washing out the material from underneath and allowing sinking to take place.

The box caisson used for the foundation of the Sutherland's River bridge, Nova Scotia, had a bottom composed of a double thickness of 12- by 12-in. timbers laid close, the timbers of the upper course running at right angles to those of the lower course. The sides were formed of vertical studding, horizontal sheathing, and diagonal bracing. This caisson, which was built on shore and made

watertight, was launched and towed to the site, after which the permanent masonry was placed to sink it to the bed of piles on which it was to rest.

The two principal advantages possessed by the concrete box caisson over the timber caisson are as follows: (*a*) the caisson may be carried up to above low-water level, thus eliminating cofferdam work; and (*b*) it is a more durable type, especially in those waters infested by marine wood borers. Caissons made of concrete will usually prove somewhat more expensive than those made of timber.

Figure 9-2*a* shows the type of caisson used as breakwaters and piers for a bridge forming a yacht landing at Glen Cove, Long Island.[1] To make the launching easier, the caissons were built in two sections. They were reinforced for exterior pressures which the structures would meet in sinking and for interior pressures which would occur at low tide by reason of the interior filling.

All caissons were cast standing on skids. When the concrete hardened sufficiently, a derrick scow lifted and set them in the water, where they were towed to position and sunk. Some of the upper sections were placed on the scow and lifted directly from there to the lower sections already in place. In the bottom sections a 3-in. hole was cast, which was closed while the caisson was being towed to position. When directly over the site of the foundation bed, water was let in by unplugging the hole to sink the structure, and sand and gravel were used to fill it.

For a very complete discussion of the subject of concrete caisson construction for breakwaters the reader is referred to a paper by W. V. Judson in the *Proceedings of the Western Society of Engineers*, vol. 14, p. 533. An abstract of this paper may be consulted in *Engineering News*, vol. 62, p. 34, July 8, 1909.

9-3. Single-wall Open Caissons. An open caisson is a boxlike self-contained structure either partly or entirely open at both top and bottom. It forms an integral and permanent part of the pier.

The open caisson which is one of the most important classes of structures used for subaqueous work, and has the distinction of being employed for the deepest foundations, may be divided into the three following types: (*a*) the rectangular single-wall caisson, consisting of a frame with solid walls and without top, bottom, interior chambers, or cutting edges; (*b*) the cylinder caisson, consisting of open cylinders of iron or masonry; and (*c*) the rectangular caisson

[1] From paper on Reinforced-Concrete Pier Construction by Eugene Klapp, *Trans. A.S.C.E.*, vol. 70, p. 448, December, 1910.

with dredging wells, consisting of a structure partly closed both at the top and at the bottom, with open wells running vertically through it. Type *a* is used where little or no sinking is required, whereas *b* and *c* are employed where sinking is necessary, *b* usually having cross sections much smaller than the others.

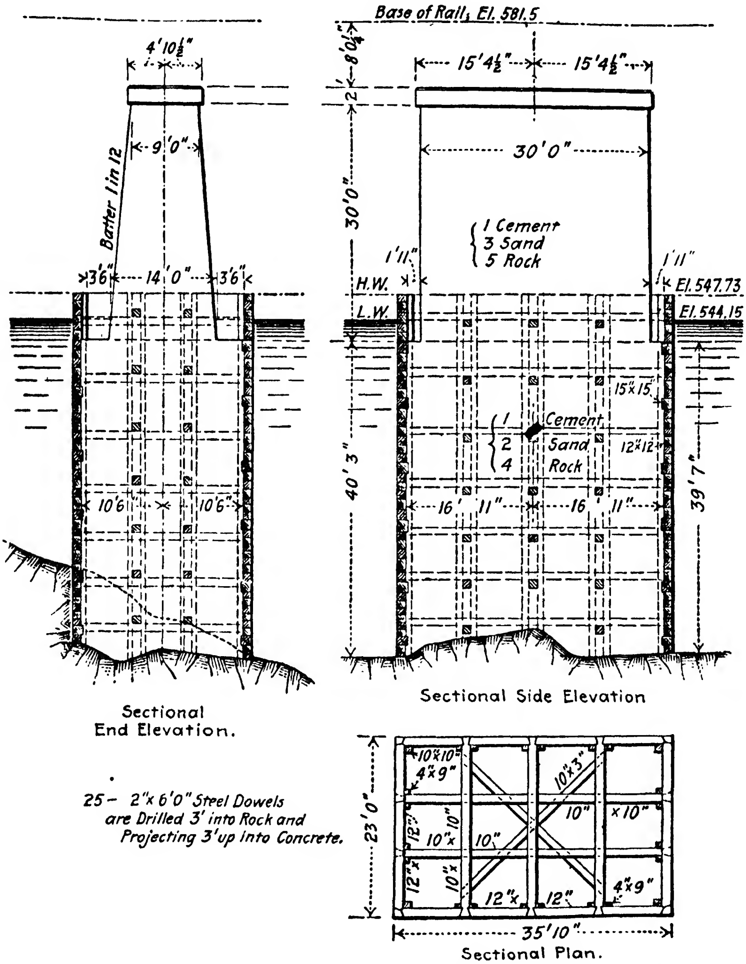


FIG. 9-3*a*.—Open Caisson for Canadian Pacific Railway Bridge over French River.

The rectangular single-wall caisson usually consists of a solid framework of 12- by 12-in. timbers thoroughly calked. It is used only where little or no sinking is required or else where the material to be sunk through is very soft, because sinking must be done by artificially weighting the structure with removable material, such

as iron rails. If soft material covers the site, as much of it as possible should be dredged out before placing the caisson. Removing the material from within the caisson after it is placed, and also using the water-jet along the sides, will greatly facilitate sinking. On reaching its final position concrete is deposited through the water to a depth of several feet and allowed to harden. This virtually forms a box caisson which is then pumped out and filled with concrete, placed in the dry, to make the foundation for the pier. It is customary to add a cofferdam on top of the caisson so that the latter may not extend above low water.

Figure 9-3*a* shows the details of the caisson used for one of the piers of the French River bridge of the Canadian Pacific Railway. The lower part was built on shore, the structure then being launched and completed in the river. Rolls of canvas were attached to the inner faces along the bottom, and, as soon as the caisson was lowered to the bottom, divers went down, spread out the canvas, and laid bags of cement on it to close the openings under the walls. A layer of mortar was then deposited through the water on the rock bottom, after which the remainder of the caisson was filled with concrete. The caisson was surmounted with a cofferdam of exactly the same construction as the caisson.

Figure 9-3*b* shows the details of the caissons used in the substructure of the Columbia River bridge of the Oregon-Washington Railroad and Navigation Company. The river bed was composed of very firm soapstone, overlaid in places with cemented boulders, gravel, and sand. The maximum depth of water at the usual stage of the river was about 30 ft., with a maximum velocity of current of 7 m.p.h.

When the caissons were framed, they were floated to place and sunk by loading with steel rails, the latter being placed in the racks shown on the drawings. All concrete was placed through the water, no attempt being made to pump out the caisson. Upon the completion of the concreting, all timbers above low-water level were removed.

The piers of the Interstate bridge over the Columbia River at Portland were founded on caissons, 16 by 57 ft., resting on pile foundations, as shown in Fig. 9-3*c*. The bed of the river consists of sand, extending to a great depth, which is subject to considerable scouring action. The cribs were sunk from 20 to 25 ft. below the river bed, and the piles were jetted down inside the crib after the material had been excavated. Concrete was deposited through the water, and when this had hardened, the water was pumped out,

the piles cut off a short distance below low water, and the remainder of the concrete placed in the open.

The concrete placed under water was deposited through a tremie consisting of a 10-in. wrought-iron pipe with a 2-cu. yd. hopper at the top. After the first charge of concrete was placed, the lower end of the tremie was constantly immersed in the soft concrete, the full length of the pipe being kept filled with concrete. On filling the hopper, the tremie was lifted slightly until the pressure was

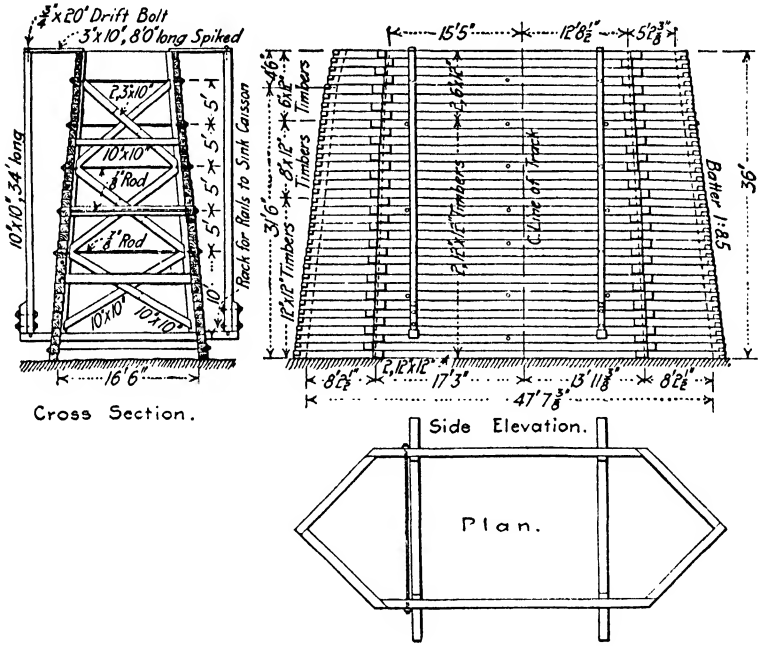


FIG. 9-3b.—Open Caisson for Piers of Oregon-Washington Railroad and Navigation Company over Columbia River.

sufficient to force the concrete out; when the hopper was nearly empty, the tremie was lowered again.

The piles were so embedded in the caisson that they formed an integral part of it; hence the pier would still be stable even though some unusual scour extended below the bottom of the base.

Figure 9-3*d* illustrates an example of the circular caisson of a form called the "basket crib." This type of caisson has been used by the Engineering Department of Boston, Mass., in a number of cases for the foundations of pivot piers. This caisson, which was 60 ft. in diameter and 40 ft. high, is of special interest on account of the cylindrical chamber, 30 ft. in diameter and 22 ft. high, in

the upper part of the caisson, which reduced the volume of concrete. The basket crib, or form for the pier foundation, was built of about 145 horizontal courses of 3- by 12-in. yellow pine planks, 8 ft. long, laid flat, and breaking joints. The ends were beveled to make radial joints, and each plank was secured to those below it by 1-in. oak

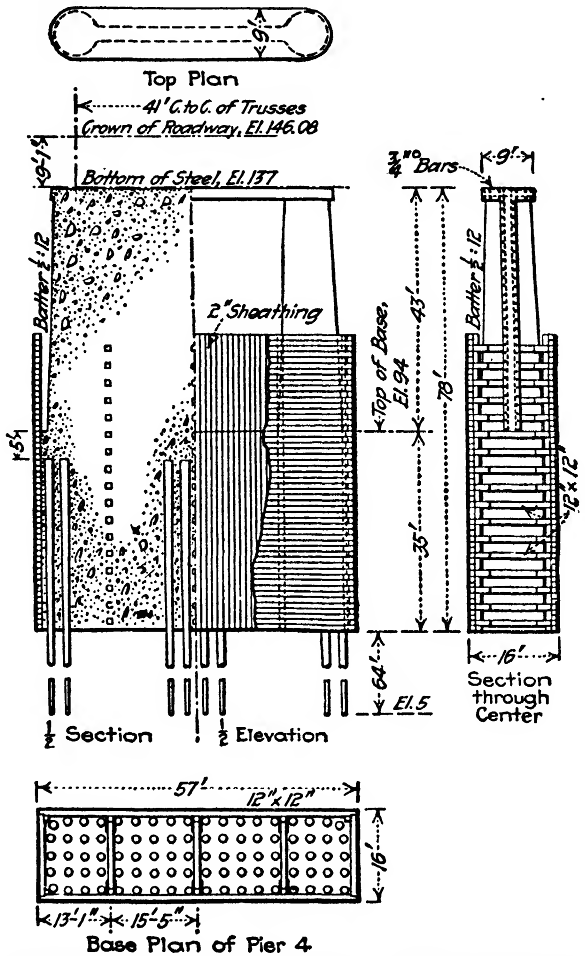


FIG. 9-3c.—Timber Caisson of Interstate Bridge across Columbia River Ore.

tree nails 9 in. long, two at each end of each plank. In addition, the planks were well spiked to the lower courses throughout their entire length with 6-in. spikes. The courses were also secured together by 4- by 12-in. vertical planks opposite alternate joints that were fastened to the inner circles of the crib by lag screws.

Before placing the caisson, the site was dredged to rock. The caisson was sunk by loading with old iron and stone and by hanging heavy chains over the walls.

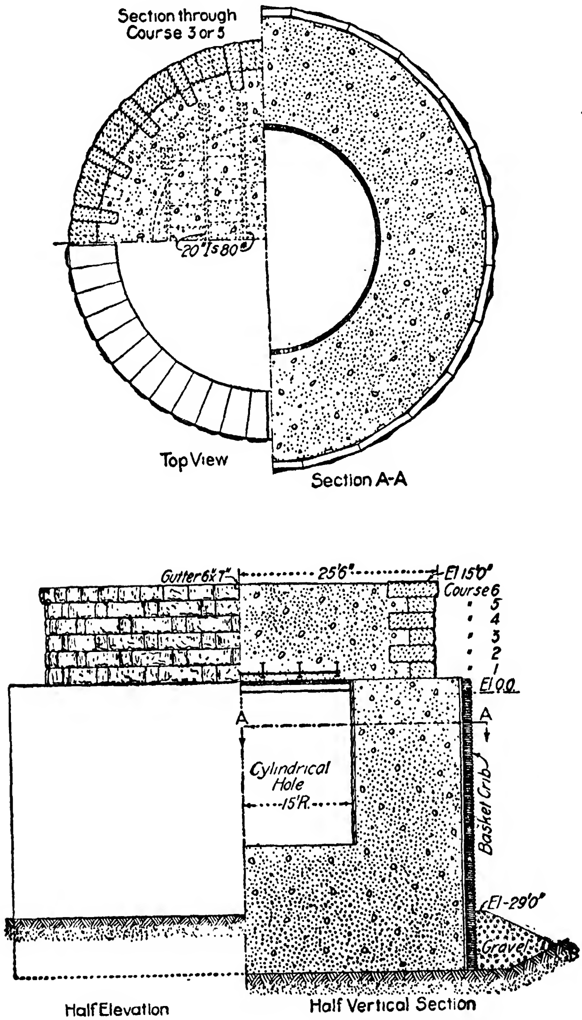


FIG. 9-3d.—Basket-crib Type of Open Caisson.

Reinforced-concrete caissons were used in the construction of a bridge over the Platte River near Bridgeport, Neb., as illustrated in Fig. 9-3e. Owing to the shallow water, the caissons were built in place, with 12- by 12-in. timbers for the cutting edge, two to five courses being used, depending on the depth of the water. The

boxes were sunk by excavating with a clamshell bucket and by loading with concrete piles, as much as 160 tons being used in some cases.

After a caisson had reached its proper depth, reinforced-concrete piles were sunk by jetting, their tops being about 5 ft. below the top of the caisson. After all the sand in the caisson around the piles had been removed by an hydraulic sand ejector, a 4-ft. sealing course was placed by means of a tremie. After this concrete had set for 24 hr., the box was unwatered and the rest of the concrete placed.

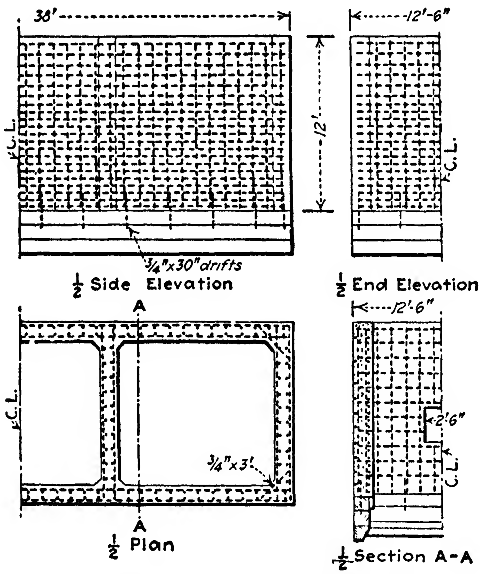


FIG. 9-3e.—Open Caisson of Concrete.

9-4. Cylinder Caissons. The cylinder caisson consists of a cylindrical shell of masonry, wood, iron, or reinforced concrete, shod with some form of cutting edge, and is sunk by excavating the material within the caisson and at the same time weighting it, or using the water-jet around the sides to decrease the friction. Other methods sometimes used include jacking, driving, and drilling. Where the cylinder is of large diameter, there may be two shells, an outer and an inner one, the space between the two being filled with concrete as the caisson sinks. Where the cylinder caisson is used, it is customary to construct the piers as an upward extension of the caisson.

This type of foundation is widely employed where the loads to be supported are not great but where it is necessary to go down a

considerable distance to avoid scouring action. It has been widely used particularly in the British provinces of the Far East, for there the rivers are dry, or nearly so, for a large part of the year, but deep and torrential during certain months, thus requiring the foundations to be bedded at a depth below that of any possible scour.

For many centuries the natives of East India have employed the masonry caisson, or "open well," as it is more frequently called, in sinking the foundations for their bridges. In their most primitive form these caissons consisted of wells large enough for but one man to work in—about 3 ft. in diameter—and were built of brick masonry resting on wooden curbs. They were sunk to a maximum depth of

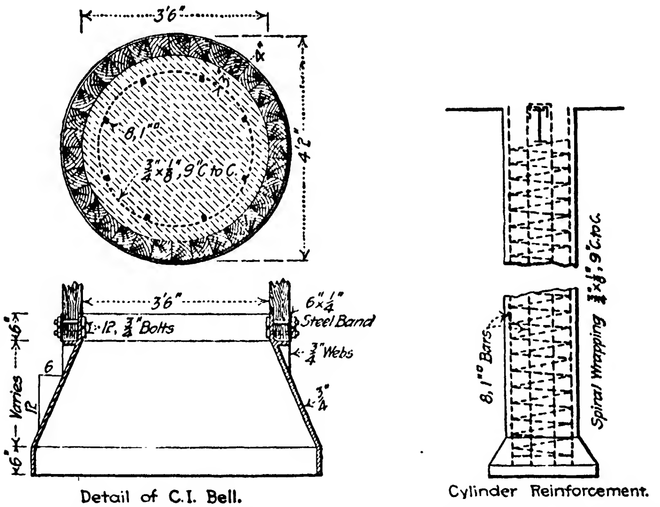


FIG. 9-4a.—Cylinder Caisson for Foundations of Shipping Pier at San Francisco.

about 17 ft. by divers excavating inside of them and bringing up the excavated material in buckets. When bedded on a firm stratum, they were filled with masonry. For those streams which were low or dry for much of the year, this was a cheap and effective method of placing the foundations.

A modern example of this general type is found in the construction of the north abutment caissons of the Chittravati bridge, where brick caissons on wrought-iron curbs were used, the exterior diameter being 12 ft. and the thickness of the brick wall 2 ft. They were sunk to a maximum depth of about 63 ft. by dredging through the wells and by loading with iron rails.

Cylinder caissons of wood are not widely used. Figure 9-4*a* illustrates the caissons of wood used for the foundations of a shipping

pier at San Francisco. Each caisson was made of Douglas fir timber staves 4 in. thick, banded with iron. The bottom consisted of a cast-iron bell attached to the staves as shown in the diagram. The cylinders were bedded about 12 in. in a hard clay stratum which was about 40 ft. below mean low water and 46 ft. below mean high water. An average depth of about 15 ft. of soft mud overlaid the clay.

The caissons were sunk through this soft material by means of four water-jets playing around the bottom of each. Little driving was necessary until the clay stratum was reached. Special frames were used on top of the caissons to receive the blows of the hammer. As soon as the desired penetration was obtained, the water and mud were pumped from the caisson—the clay effectually sealing the bottom—and the bottom carefully inspected, after which the reinforcement was placed and the cylinder filled with 1:2:4 concrete.

9-5. Metal Cylinder Caissons. The shell may be of cast iron, wrought iron, or steel, the last being used almost exclusively in this country. The metal type possesses the three following advantages over the masonry: (*a*) greater strength, (*b*) a higher degree of water-tightness, and (*c*) less friction developed in sinking. After the caisson is sunk to a proper bearing, it is filled with concrete or sand, the former being invariably used in America, whereas English engineers use the latter to a considerable extent.

The California City Point coal pier offers a good example of the use of cylinder caissons of small diameter, being formed of flanged cast-iron pipe 4 ft. in diameter. The depth of water at the site was about 30 ft., but from 4 to 40 ft. of mud overlaid the hard bottom on which the caissons were to rest.

In order to increase the bearing area on the bottom a special conical section was made for the lower end of the cylinders, the maximum diameter of this section being 8 ft. The shell above this lower section was composed of regular 4-ft. cast-iron pipe in 12-ft. lengths, each section being fastened to the one above and below with 44, $1\frac{3}{8}$-in. bolts through the flanges.

Sinking the shells was accomplished as follows: A number of sections were bolted together, lowered vertically to position on the mud bottom, and braced there with guys. They were sunk by dredging out the inside of the pipe by means of a $\frac{1}{2}$-cu. ft. orange-peel bucket, new sections of pipe being added as the cylinder went down. Where the resistance to sinking was considerable, the work was facilitated by temporarily loading the top of the caisson with steel beams and girders and by the use of the water-jet around the

cutting edge. After sinking operations were completed, the conical portion of the cylinder was filled with concrete deposited through the water, and, when the concrete had hardened, the water was pumped from the cylinder. Fourteen vertical reinforcing rods of $1\frac{3}{16}$-in. diameter were then placed in each cylinder, and the latter was filled with a 1:3:5 mixture of concrete, rammed in 12-in. layers, to form the foundation for one of the columns of the coal pier.

The caissons for the highway bridge across the Kansas River at Fort Riley are typical examples of steel cylinder caissons (so much used years ago for light highway bridges) where it was necessary to go down some distance below the bed of the stream to get proper bearing material. Each pier consisted of two cylinders well braced together, each cylinder being 5 ft. in diameter. The metal used was $\frac{1}{4}$ in. thick, although a thickness of $\frac{3}{8}$ in. would have been better. The cylinder sections were in 6-ft. lengths, were butt-jointed with splice plates on the inside, and were riveted up in 12- and 18-ft. sections in the bridge shop. The cylinders were 54 ft. long and were sunk through fine sand by dredging and weighting, at a time when the river was dry, to an average depth of 24 ft. below the river bottom.

One of the most expensive items connected with the sinking of cylinder caissons is that of artificially weighting the structure to promote sinking, because this weighting material must be removed and replaced each time new sections are added to the caisson. Largely for this reason, where the size of the caisson will permit, it is advisable to use a double wall, so that much of the permanent concrete filling may be placed during sinking and thus decrease the amount of temporary loading necessary.

This was done in the caisson for the pivot pier for the Omaha Bridge and Terminal Company's bridge across the Missouri River from East Omaha to Council Bluffs, Iowa. As shown in Fig. 9-5*a*, the caisson, which was of steel, had an outer diameter of 40 ft. and an inner diameter of 20 ft. It rested on solid rock 120 ft. below low water. For the first 50 ft. the material was sand and clay, and below this there was about 60 ft. of coarse sand overlying a few feet of boulders which rested on solid rock. At a low-water stage of the river the depth of water was slight. Sinking was accomplished by a combination of three agencies: dredging the material from inside the caisson, using water-jets to reduce the side friction, and filling the space between the two shells with concrete. On completing the sinking, the well was also filled with concrete. The details of the caisson are clearly shown in the illustrations.

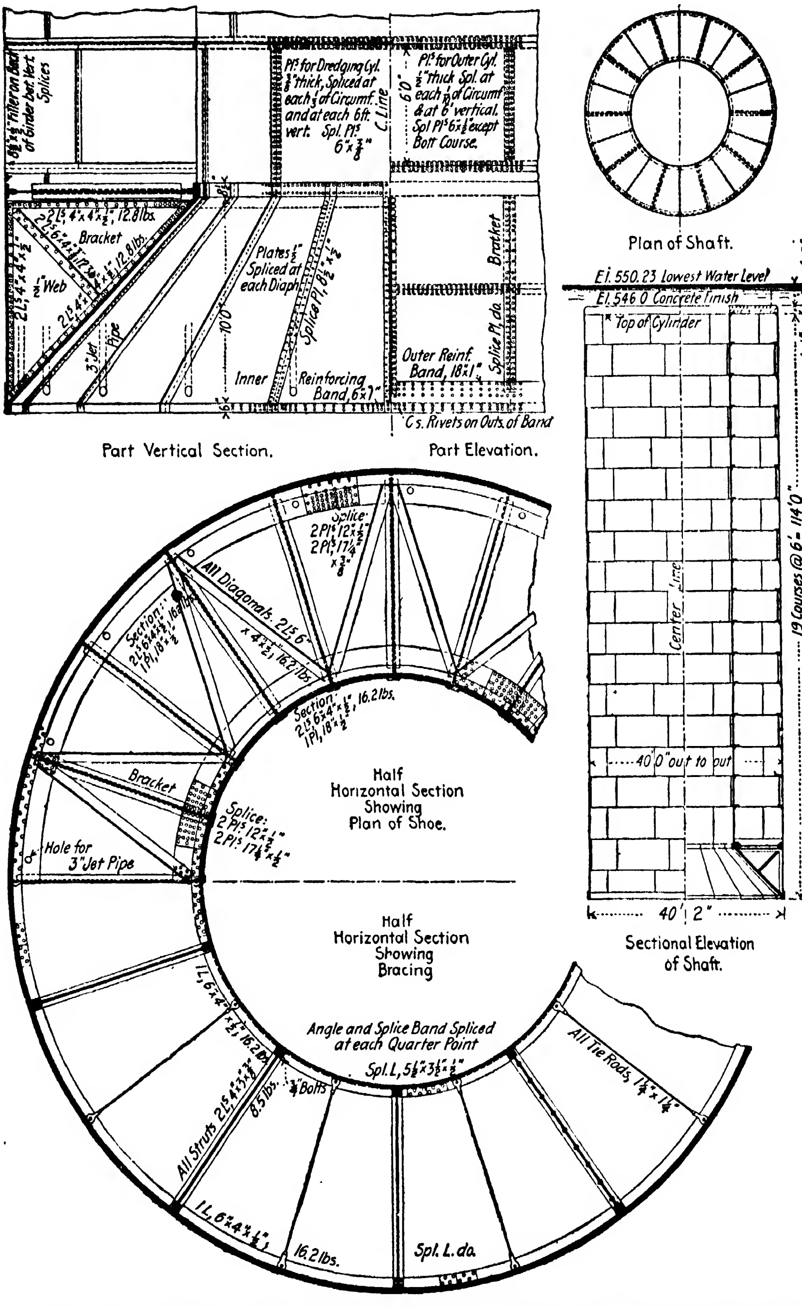


FIG. 9-5a.—Details of Open Caisson for Pivot Pier of Omaha Interstate Bridge.

In sinking the 44-ft. diameter double-wall caissons for the Atchafalaya River bridge at Morgan City, La., in 1933, to a depth of 176.5 ft. (which was then the record depth for foundations) considerable difficulty was encountered because of the lack of soil friction. The water was about 53 ft. deep, and the river bottom consisted of a 30- to 40-ft. depth of semifluid silt. Twenty-nine feet

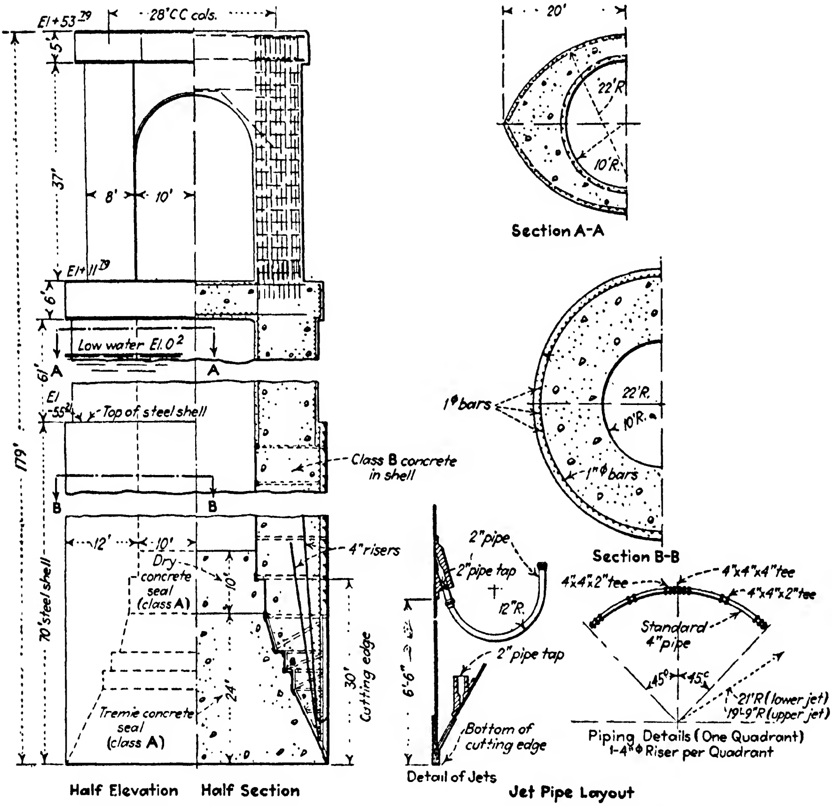


FIG. 9-5b.—Metal Cylinder Caisson for the Atchafalaya River Bridge at Morgan City, La.

of concrete in the caisson placed the cutting edge on top of the silt, and the caisson sank 45 ft. in this silt before concrete could be brought up in the shell to water level. The pneumatic method (Chap. X) was used for part of the sinking. When the sinking was completed, the 20-ft. diameter well was concreted to a depth of 24 ft. with concrete placed through water. After this had hardened, a 10-ft. reinforced-concrete seal was placed. The rest of the well

was later flooded. Figure 9-5*b* illustrates a typical caisson of this bridge but not the deepest one.

9-6. Metal Cylinder Caissons for Buildings. One of the few applications of the open-caisson method of placing building foundations is found in the 71-story Manhattan Company Building (New York City),[1] built in 1929. This building replaced a number of small buildings. The construction procedure decided on was to sink metal cylinder caissons under the existing buildings; but only part-size caissons—large enough to carry 20 stories of steel—were to be used, so that steel erection could be begun quickly and the caissons completed later to full-size load. The open-caisson method was used, since the space in the old basements and the limited time made the pneumatic method (Chap. XI) impracticable.

The soil was fine sand and clay (typical Manhattan quicksand) to a depth of about 49 ft. below street level and then a hardpan mixture of hard gray clay, gravel, and boulders to rock at an average depth below street grade of 64 ft. Water level was 22 ft. below the average street grade. The old basement floor was about 10 ft. below street grade and the new basement floor about 21 ft. below.

The cylinders, 44, 48, and 52 in. in diameter and $\frac{1}{2}$ to $\frac{5}{8}$ in. thick, were for the most part jacked down, the hydraulic jacks acting against I-beams notched into the masonry walls and piers of the old buildings. The hydraulic rams operated at pressures up to 6,000 lb. per sq. in., or a total pressure of 300 tons.

The cylinders were excavated by hand, extreme care being taken to keep the bottoms of the cylinders well below the level of excavation at all times, so that no boiling of quicksand would occur. At the first evidence of boiling, gravel was quickly dumped into the caisson. Water was removed by steam siphons discharging into settling tanks. The excavated material was hoisted in buckets by electric winches.

On reaching hardpan, the material was carefully cut out under the edge of the cylinder, which was then jacked down into the hardpan from 1 to 3 ft. to cut off the water. The excavation was carried down as a shaft to hard rock. Sheeting was used in the hardpan only where disintegrated rock was met.

Considerable difficulty was experienced in sinking the cylinders through the lower part of the sand owing to the presence of boulders. Where these were encountered, there was maintained all the jacking pressure which the cylinders would stand and for which reactions were available under the old building. In several cases, where it

[1] See *Eng. News-Record*, vol. 104, p. 691, Apr. 24, 1930.

was impossible to dislodge the boulders from under the cutting edge, steel sheeting made of pieces of steel plate bent to the radius of the cylinder was driven. Before it was driven the sheeting was burned out to fit around the obstruction, the shape of which was determined by sounding with bars. In one instance the cylinder itself was burned out to fit around a boulder at the top of the hardpan.

Six of the cylinder caissons were driven by pneumatic pile-hammers because the wrecking of the old buildings proceeded so fast that jacking was not possible. The excavation in the cylinders and in the hardpan was done in the same manner as in the jacked cylinders.

When demolition of the old buildings had progressed far enough to permit, steam shovels started excavation, the water being lowered by continuous pumping. Meanwhile the cylinders were filled with $1:1\frac{1}{2}:3$ concrete, and steel billets were placed on top, the billets being larger than the cylinder piers supporting them.

As soon as excavation reached subgrade, the cylinders installed as partial-load foundations were enlarged by putting down around them square caissons. Steel sheet-pile boxes were driven around the cylinders from subgrade to hardpan. These were excavated and the shafts continued without sheeting down through the hardpan to rock. The supplemental caissons were then filled with concrete.

9-7. Metal Cylinder Caissons Placed by Boring. One of the more recently developed methods of sinking steel-cylinder caissons is by forming a saw-toothed cutting edge on the bottom of the cylinder and then boring the cylinder into position. Figure 9-7*a* illustrates the machine used in sinking 112 caissons from 4 to $8\frac{1}{2}$ ft. in diameter and about 66 ft. long, the thickness of shell being from $\frac{3}{8}$ to $\frac{1}{2}$ in., for the new Federal Building in lower Manhattan, New York City. The teeth were cut with a gas torch, alternate teeth being bent inward and outward to give a set such as to produce a kerf twice the thickness of the cutting-edge plate. To prevent wear of the cutting teeth, their exposed surfaces were coated with a thin layer of tungsten carbide applied by gas welding.

The rig for sinking the caissons consisted of an all-welded structural-steel tower 78 ft. high, carried on a welded bedframe. Two of the drums of the electric hoist were used to lift the cylinders into position by means of a short-boom derrick. The third drum, which had a lifting capacity of 70 tons, served to handle the rotating head assembly which fitted into the leads of the tower.

The 125-hp. electric motor drove the rotor head through a 220 to 1 worm-gear reducer and developed a torque of 1,000,000 ft.-lb. for rotating the cylinders. The rotor head was equipped with driving lugs to engage lugs on the cylinder headplate. The speed of rotation could be varied from $3\frac{1}{2}$ to $11\frac{1}{2}$ r.p.m. The rate of

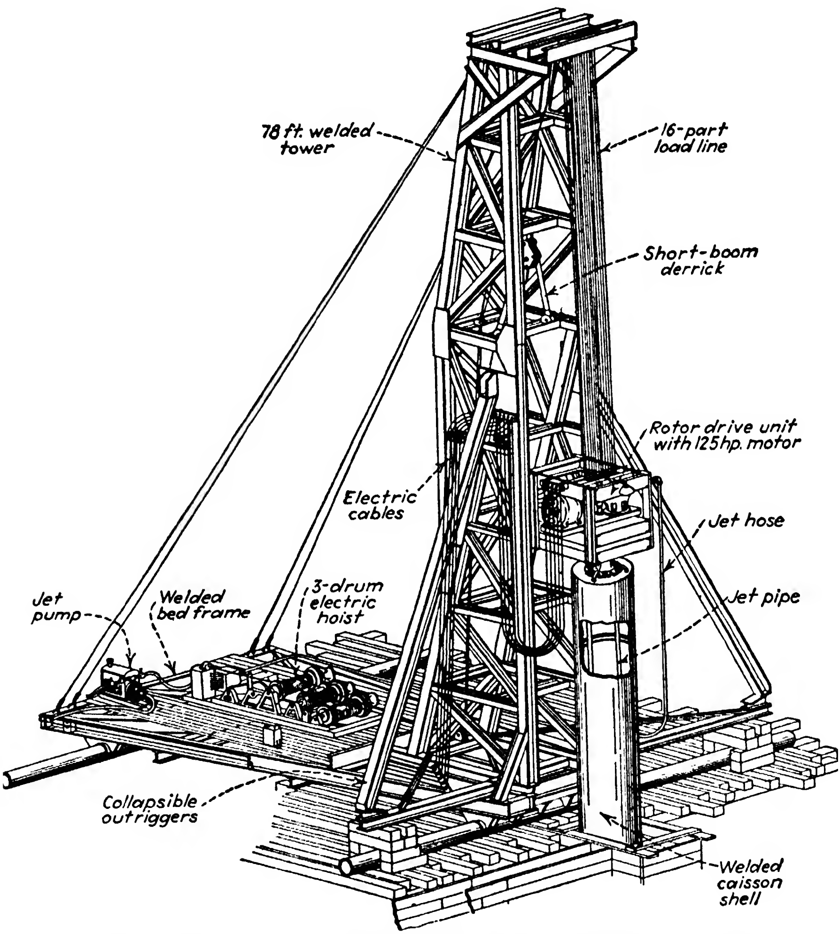


FIG. 9-7a.—Machine Used for Boring Metal Cylinder Caissons into Position.

sinking varied from 12 to 20 ft. per hr., resulting in sinking the caissons in 4 to 6 hr., as compared with the usual 6 to 8 days required when using compressed-air methods. Before rotation was begun, the jet pump was started in order to partially fill the shell with water. A hydrostatic head was built up to force the water out

underneath the cutting edge as the cylinders were rotated, with a slightly eccentric motion.

The jets carried away from 65 to 75 per cent of the material within the shells. As soon as a caisson reached rock, the headplate was removed and the remaining material was taken out by a small clamshell or orange-peel bucket. The final cleaning off of the bottom was done by hand after the water had been pumped out. Where there was any indication that the shell was not sealed into rock and that further excavation and pumping would result in a blow-in, an air lock was placed on top, after which the pneumatic-caisson method was used. A seal of concrete 25 ft. deep was placed while the caissons were under air. For the caissons that did not require air to seal, the concrete was placed to a depth of 10 ft. with bottom-dump buckets. After this hardened, the cylinders were pumped out and the remainder of the concrete filling was placed in the dry.

9-8. Reinforced-concrete Cylinder Caissons. One of the early (1910) examples in America of the application of this type of caisson was in the foundations of the pedestals for the Penhorn Creek viaduct of the Erie Railroad. A single caisson was used for each pedestal. The shell of the caisson consisted of a hollow reinforced-concrete cylinder, having an exterior diameter of $6\frac{1}{2}$ ft. and an interior diameter of $4\frac{1}{2}$ ft., thus giving a thickness of 1 ft. It was reinforced with $\frac{1}{2}$-in. vertical rods, spaced 9 in. center to center, and by $\frac{1}{2}$-in. horizontal circular rods spaced 6 in., the former located 2 in. from the outside face and the latter just inside of these. The depths to which the caissons were sunk varied greatly, many of them extending to about 70 ft. below the surface of the ground, which corresponds to about 55 ft. below ground-water level.

In constructing a caisson, a pit about 11 ft. square and 10 ft. deep was excavated and lined with 3- by 12-in. butt-jointed sheathing, braced by 12- by 12-in. horizontal rangers. Four vertical 12- by 12-in. sticks were then placed, one at the middle of each waling piece, to serve as a guide for the caisson. The cast-iron cutting edge, shown by the heavy lines in Fig. 9-8*a*, was placed in the bottom of the pit. Above this were placed outside and inside collapsible steel forms in 5-ft. lengths. All caissons were cast in 20-ft. units, the caisson being built to this height, allowed to set, sunk, and another section added, the whole operation being repeated until the desired depth was reached. Each section was allowed to harden 6 days before it was sunk.

Sinking through the mud and sand was effected for the most part by interior excavation with an orange-peel bucket. The water-jet was used to some extent and weighting was also resorted to at times. It was found advantageous to keep the jet pipes separate from the caisson and to work them by hand. The average rate of sinking through mud was $6\frac{1}{2}$ ft. per day, while through the dense underlying sand only about $1\frac{1}{2}$ ft. per day could be accomplished.

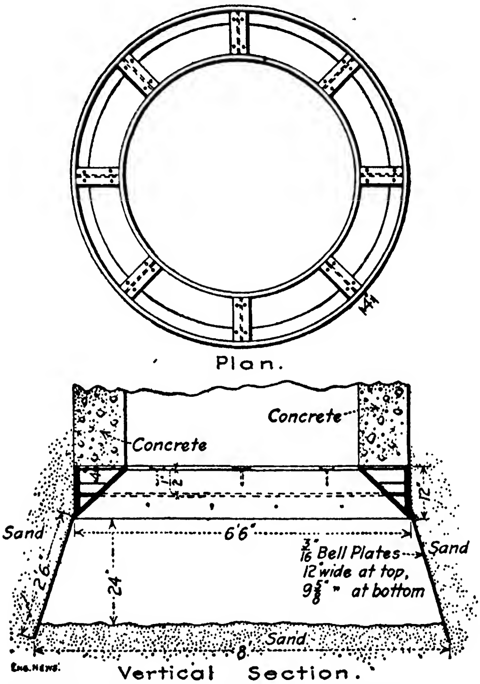


FIG. 9-8*a*.—Cutting Edge of Caisson, Penhorn Creek Viaduct, Jersey City, N. J.

It was at first intended to found the caissons on rock, using an allowable bearing pressure of 10.8 tons per sq. ft., but later, owing to the greater depth of the rock, it was decided to found them on the dense sand above, which it was thought would safely bear a load of 7 tons per sq. ft. In order to reduce the unit pressure to this amount, the bottom was belled out as shown in the illustration. The conical or belled section, which consisted of a number of $\frac{3}{16}$-in. steel plates, was placed by a diver, who, with the aid of a water-jet, forced the dense sand from around the cutting edge and placed the plates. Each plate was forced into the sand a slight distance at the bottom and sprung behind the cutting edge at the top. Upon

the completion of this work, the caisson was filled with 1:2½:5 concrete.

Figure 9-8b shows details of the caisson used by the Chicago, Burlington, & Quincy Railroad in the construction of a bridge over the Platte River near Ashland, Neb. These caissons were sunk about 50 ft. One disadvantage of this type of construction is the length of time involved in waiting for the concrete to harden after each section is poured. To eliminate this feature, cylinders may be made of precast sections, with suitable devices for connecting the reinforcement of the various sections.

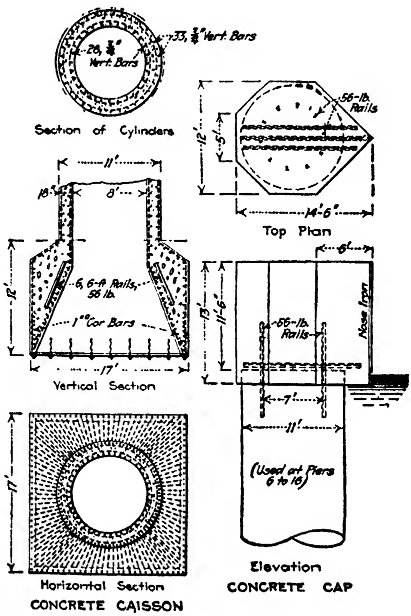


FIG. 9-8b.—Reinforced-concrete Cylinder Caisson.

9-9. Rectangular Open Caissons with Dredging Wells. This type is based on the same principle as the double-wall open-cylinder caisson, differing from the latter only in the matter of shape and size. Perhaps the most notable difference lies in the fact that the open-cylinder caisson has but one dredging well, while the type to be treated in this and the three following articles always has more than one.

The open caisson with dredging wells is a type of construction which has been employed for the deepest foundations ever used for bridge piers. Theoretically, there is no limit to the depth to which this class of caissons may be sunk. The essential principle

of the construction is a boxlike structure of wood, iron, or reinforced concrete, with ballast pockets in the same and with open wells running vertically through it, these wells flaring out at the bottom to practically the whole area of cross section of the caisson. Through the wells, by means of dredges, the material is excavated from the bottom, and this, together with simultaneously filling the pockets with concrete, is usually sufficient to sink the structure. As in the other forms of caissons, when the structure is sunk to good bearing material, the wells are filled with concrete.

The great advantage of this type of structure for foundations is that all the work is done above water, so that the cheapest class of labor can be employed, thus under favorable conditions making a very economical foundation. In the use of the open-caisson method there are three disadvantages, which are absent in either the cofferdam or the pneumatic-caisson process. They are as follows: (*a*) the character of the bottom on which the caisson finally rests can never be as satisfactorily known as when it is possible to exclude the water and inspect the bottom in the dry, nor can the latter be leveled and cleaned as easily as when the other methods are employed; (*b*) the concrete which is placed in the bottom of the well must be placed through the water and consequently is not so good concrete as when placed in the dry; and (*c*) it is difficult to estimate the possible rate of sinking owing to the trouble which boulders and sunken logs will offer when encountered under the cutting edge. In spite of these disadvantages, the open caisson with dredging wells is widely employed for depths greater than can be satisfactorily handled by the cofferdam process and where the cost of the pneumatic-caisson process prohibits its use, or where the depth is greater than 110 ft. below water level, the limiting depth in practice for pneumatic-caisson work.

Since the sinking of caissons with dredging wells is not under thorough control, the caissons should always be made large enough to allow a moderate amount of deviation from the correct position. As the wells or dredging tubes are the chief means by which the descent can be regulated as regards direction, these should be so placed as to facilitate this objective. For example, when one end of the caisson strikes soft material and consequently sinks faster than the other end, the caisson can be brought back to a vertical position by dredging solely from the high-end well. But, on the other hand, if the wells are distributed along the longitudinal center line, as was the case in the Hawkesbury bridge caissons (Art. 9-11) and Fraser River bridge caissons (Art. 9-10), and one side strikes softer material

than the other side, it becomes difficult to keep the structure from tilting. A better arrangement, consisting of two longitudinal rows of wells, was used in the Willamette River bridge caissons (Art. 9-10).

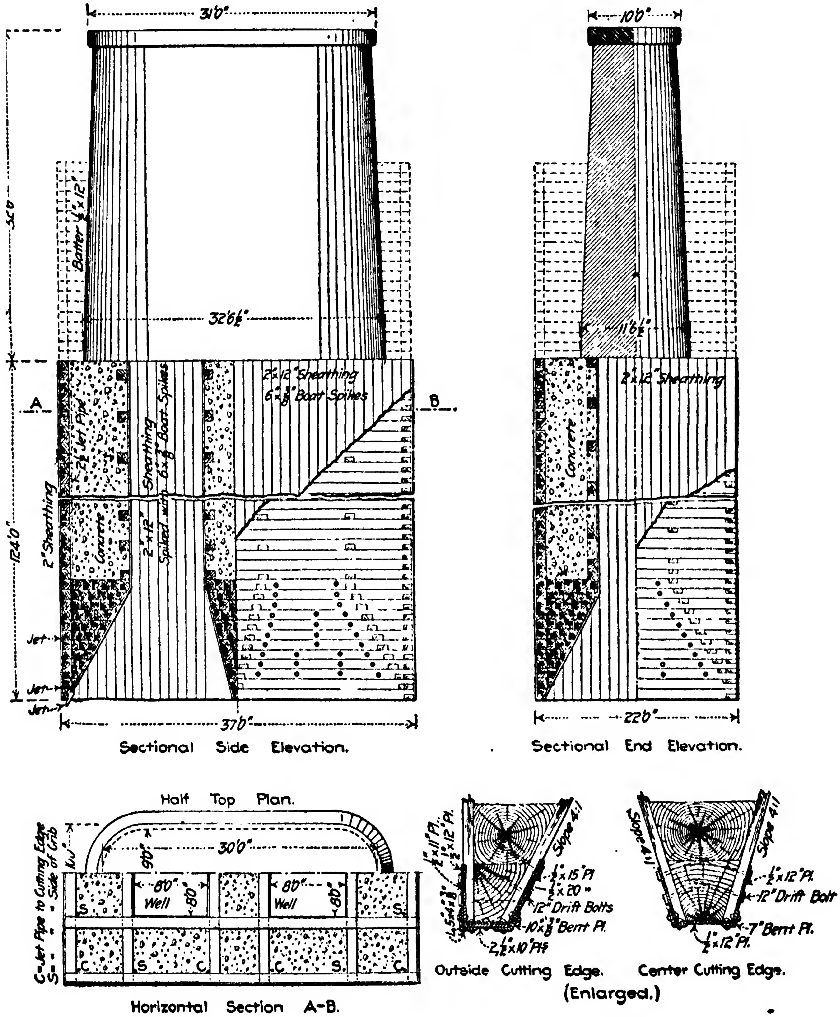


FIG. 9-10a.—Open Caisson for Pier 4, Fraser River Bridge, New Westminster, B. C.

9-10. Construction with Timber. Figure 9-10a shows the details of the deepest caisson, that for Pier 4, of the Fraser River bridge at New Westminster, British Columbia. This caisson was of timber and was sunk to a depth of 135 feet below water level.

The outside walls were built of solid courses of 12- by 12-in. timber, sheathed on the outside with vertical 2-in. planks. These walls were built on a solid triangular-shaped timber base of 12 courses of material, the inside of this base also being sheathed with 2-in. planks. The dredging wells were framed with both longitudinal and transverse timbers, laid solid near the bottom but open above this, and were sheathed with 2-in. material. The outer- and well-wall timbers formed a series of pockets which were filled with concrete during the sinking of the caisson. All the seams, both in the horizontal timbers and vertical sheathing and in the upper two courses of the solid timber base, were thoroughly calked.

The caisson was built to a height of about 14 ft. on ways on the shore and then launched and towed to the site where it was to be sunk. Here an 8-in. layer of 1:2:3 concrete was placed on the deck and allowed to harden for a few days to increase the watertightness of the pockets. The caisson was then gradually built up, concrete being added simultaneously with the building, thus causing the structure to sink. At the start the depth of water was about 50 ft., but before the sinking was completed, this depth had increased to 65 ft., due to scour. The caisson was guided in sinking by means of long piles. As soon as the river bottom was penetrated a few feet, the concrete in the pockets was built up above water level and all but the 2-in. sheathing was omitted around the wells. The material penetrated was mostly sand and silt, and 85 per cent of this was removed by the sand- and mud-pump process (Art. 9-15). The caisson finally rested on a bed of compact gravel 135 ft. below the normal stage of the river. In sinking, the water-jet was used, the jet pipes being in the positions shown in the illustration.

On the completion of the sinking, concrete was deposited in the chamber formed by the flaring out of the wells, and this was followed by filling the wells. This concrete, to a depth of 70 ft., was deposited through the water, and the remainder was placed in the dry, the water being pumped out of the wells previous to placing the concrete.

The caisson for pier 3 of this bridge, when well down in the sand, encountered sunken logs, causing it to tilt. One of these logs had a diameter of 2 ft. and extended clear across the caisson. This was removed by boring holes in it with an auger 100 ft. long, the point being set by a diver. In these holes were placed charges of dynamite, which on exploding blew the log to pieces. The top of the caisson was surmounted with a cofferdam, as the elevation of the top was about 10 ft. below ordinary water level.

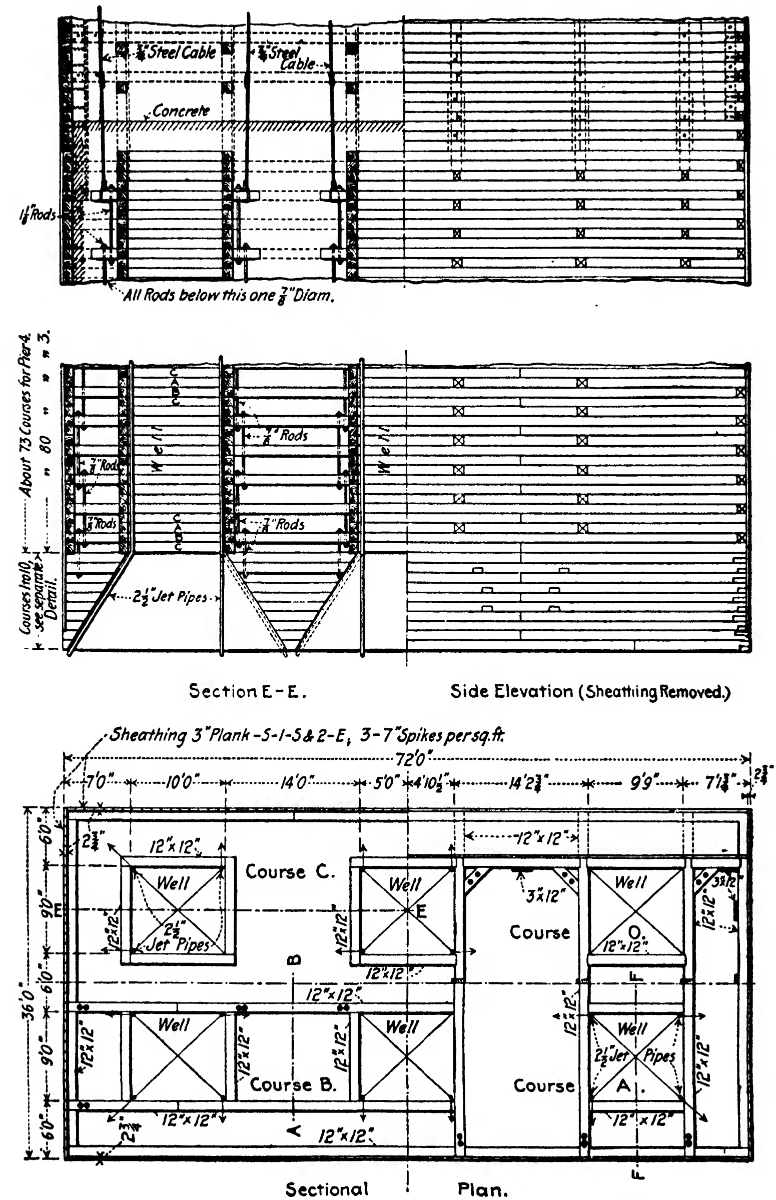


Fig. 9-10*b*.—Open Caisson and Cofferdam for the Oregon-Washington Railroad and Navigation Company's Bridge at Portland, Ore.

Figures 9-10*b* and 9-10*c* show the details of construction for the 36- by 72-ft. open caisson of the Willamette River bridge, of the Oregon-Washington Railroad and Navigation Company. There were six wells, each 9 by 10 ft. in the clear, flaring out at the bottom

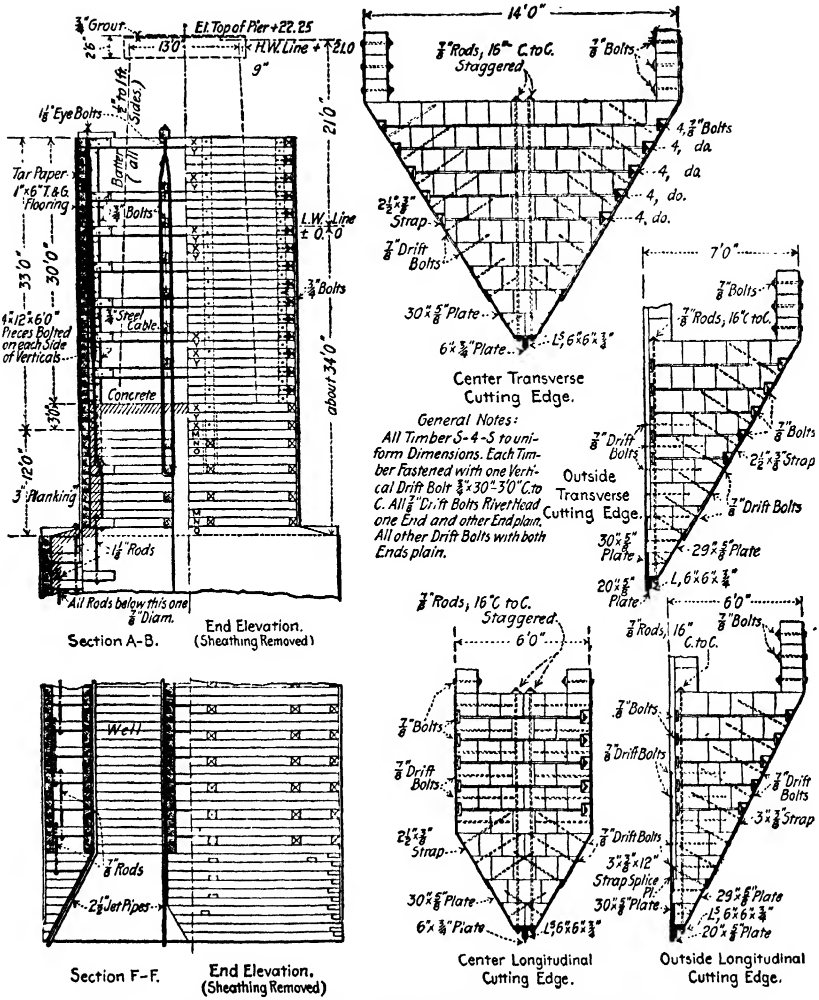


FIG. 9-10c.—Open Caisson with Dredging Wells and Superimposed Cofferdam.

to occupy the whole area of the caisson. The walls of the wells and the outside walls of the caisson were made of a single thickness of 12- by 12-in. timbers laid close, the latter being sheathed on the outside with 3-in. material. As shown in the plan, certain timbers

of the well walls were extended the entire length and breadth of the caisson to brace the same.

The lower part of the caisson consisted of V-shaped walls and bulkheads, there being two of the latter running transversely and one longitudinally. The widths at the top were 6 ft. for the longi-

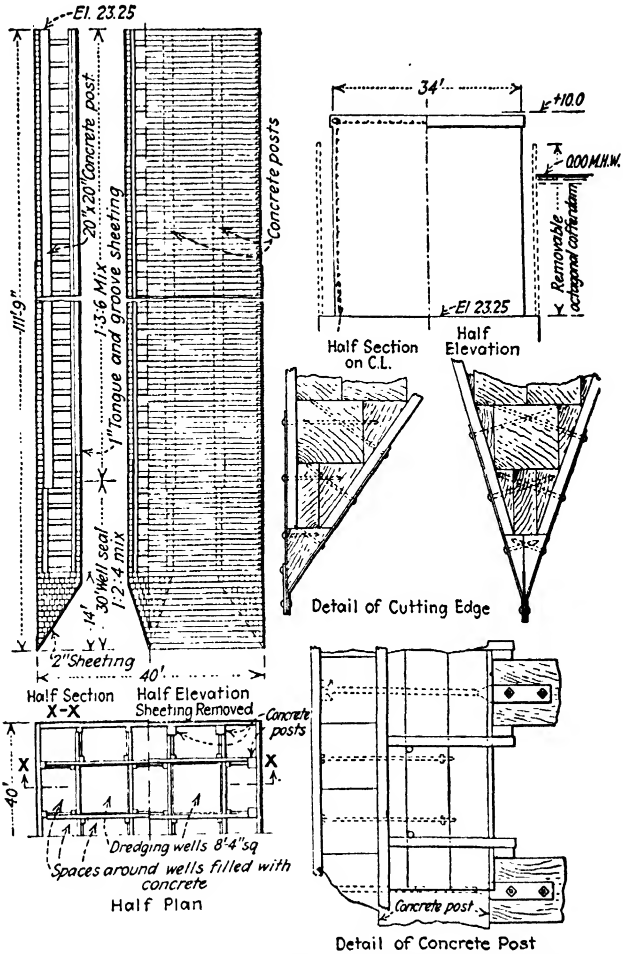


FIG. 9-10*d*.—Timber Open Caisson with Dredging Wells of the Carquinez Strait Bridge.

tudinal walls and bulkheads, 7 ft. for the transverse walls, and 14 ft. for the transverse bulkhead. In all cases the cutting edges were reinforced with steel angles as shown in the drawings.

A 33-ft. cofferdam surmounted the caisson, the walls being of the same construction as those of the caisson, except that the sheath-

ing consisted of 1-in. tongue-and-grooved material, with tar paper between it and the large timbers. The cofferdam was braced with horizontal 12- by 12-in. timbers running both longitudinally and transversely and bearing against vertical 12- by 12-in. timbers, which, in turn, took bearing against the walls of the cofferdam.

Before sinking the caisson, borings made around the perimeter of the crib showed that the surface of the good bearing stratum was on a considerable slope, a difference of 22 ft. being found for opposite diagonal corners. To level this off, pipes were sunk and holes drilled to about 2 ft. below the lowest elevation of the top of this cemented gravel stratum. Dynamite was placed in these holes and exploded, and in this way the hard material was broken up

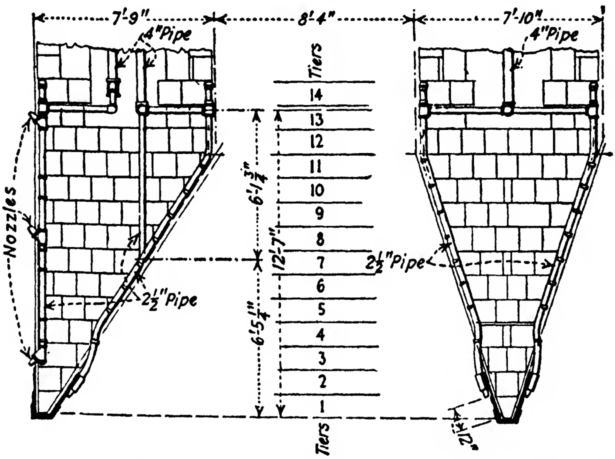


FIG. 9-10*e*.—Jet Arrangement for Caissons of the Carquinez Strait Bridge.

through 50 ft. of gravel and sand, before any excavation had been made. When the caisson reached this cemented gravel, the latter was easily removed with orange-peel buckets working through the dredging wells. The depth to which the caisson was sunk was about 130 ft. below low water, or 151 ft. below high water.

Figure 9-10*d* shows details of the caissons of the Carquinez Strait bridge which were sunk to an elevation 135 ft. below water surface. These were sunk in teredo-infested waters and to protect the timbers on the exterior and in the wells they were given a thorough spray coating of a preservative paint, made of a compound of asphalt and creosote. The timbers were next covered with a layer of impregnated tar felt, tacked on. The felt was given a spray coat, after which the outer sheathing was spiked on and, in turn, given a spray coat. It was thought that this protection would

be sufficient for at least a year, or until the structure was landed and concreted. To protect the interior timbers in case the exterior timbers were eventually eaten away, the former timbers were butt-framed against vertical concrete posts above mud-line elevation.

An elaborate system of water-jets were used to promote sinking, these consisting of outside or friction relief jets and inside or cutting

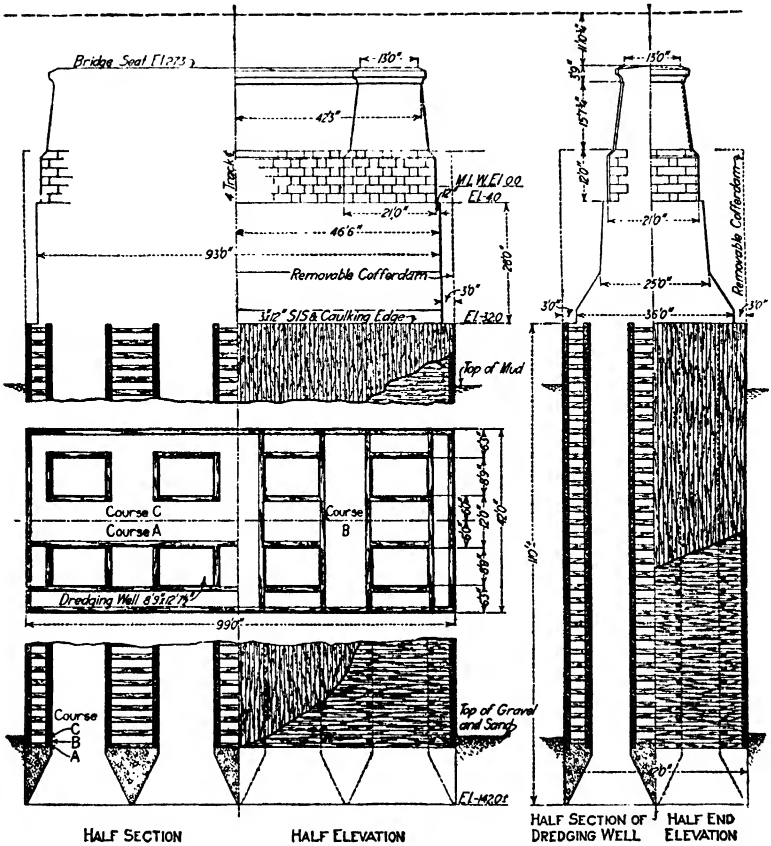


FIG. 9-10*f*.—Timber Open Caisson with Dredging Wells of New Thames River Bridge at New London, Conn.

jets, as shown in Fig. 9-10*e*. The outside jets were placed on four sides of the caisson and spaced about 6 ft. centers horizontally. The nozzles were $\frac{3}{4}$ in. in diameter. The inside jets were spaced about 10 ft. centers horizontally, the nozzles being 1 in. in diameter.

Two types of cutting edges were used as shown in the illustrations, those of the first two caissons being blunt and those in the following four being sharp. As might be expected, the sharp edges

gave a little greater sinking speed, but the blunt edges furnished better stability during sinking and landing. The average rate of sinking was 0.5 ft. per shift of 8 hr., and the maximum rate was 1.3 ft.

In the new Thames River bridge of the New Haven Railroad at New London the lower part of the caisson is of concrete and the upper part of timber, as shown in Fig. 9-10*f*. An enlarged view of the lower part is shown in Fig. 9-14*d*, which also shows the steel protection of the cutting edge.

9-11. Construction with Metal. The two advantages possessed by this type of caisson are as follows: (*a*) the speed with which the caisson may be built, and (*b*) the small space occupied by the metal, thus leaving a maximum amount of space for the concrete filling. The disadvantages are as follows: (*a*) the cost of the caisson, especially where the metal has to be transported long distances; and (*b*) the possible lack of permanency of the metal.

One of the most notable examples of the use of open caissons with dredging wells is that of the Hardinge bridge across the Lower Ganges, 120 miles above Calcutta, opened in 1915. In plan the caissons have semicircular ends and straight sides, being 35 by 63 ft. As shown in Fig. 9-11*a*, the caissons contain two dredging wells each $18\frac{1}{2}$ ft. in diameter. The well curb is built of steel and is 15 ft. 7 in. high. It is continued upward as a caisson, the distance varying in different caissons depending on the depth of water. The space between the walls is filled with concrete. Above this the dredging wells are lined with steel to give watertightness and added strength and also to serve as a form for the mass concrete. Above the steel-frame part, the walls consist of molded concrete blocks weighing about 6 tons each, these blocks being carried up to low-water level. After the caissons were sunk to their final depth, the bottom and the top of the wells were plugged with concrete and the space between filled with sand. The caissons rest on sand, the pressure being 9 tons per sq. ft., allowing for buoyancy but not for skin friction.

Another notable example of the use of open caissons with dredging wells in bridge foundations is that of the substructure of the Hawkesbury bridge in southwestern Australia, where the caissons were sunk to a maximum depth of almost 162 ft. below high water, the range of tide being 7 ft.

The caissons of the Hawkesbury bridge were oblong in plan with rounded ends, the length being 48 ft. and the width 20 ft. The lower 20 ft. was splayed out to form a tapered shoe 2 ft. wider

all around the bottom. Along the center line, parallel with its length, were three wrought-iron dredging tubes, 8 ft. in diameter and 14 ft. apart on centers, strongly braced to the sides of the caisson and to each other. At the bottom these wells splayed out in the form of a trumpet mouth to meet the outer skin and each other in a cutting edge made of steel. Between these wells and the sides of the caisson were pockets to be filled with concrete as the caisson sank.

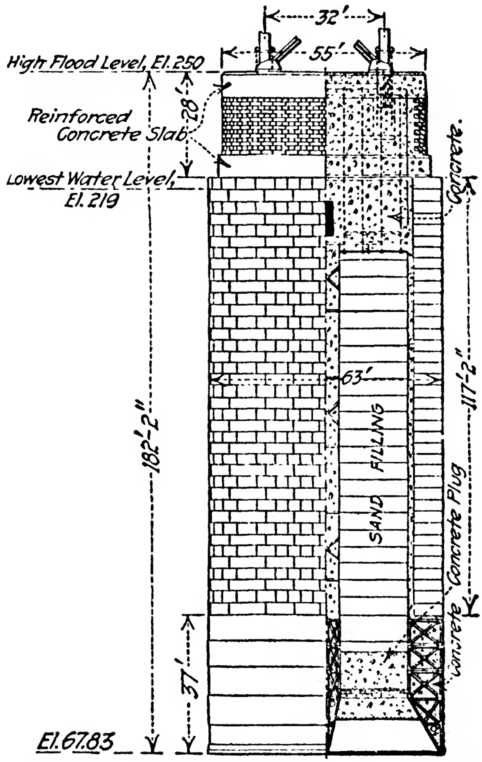


FIG. 9-11*a*.—Metal Open Caisson with Dredging Wells for Hardinge Bridge.

Sections of steel for the sides and of wrought iron for the dredging tubes were added as the caisson sank, the sinking being effected by dredging out the material under the caisson through the wells and by filling the pockets with concrete. All caissons were bedded on firm sand which was overlaid with mud and silt of varying depth, that at pier 6 being 108 ft. with a depth of water at low tide of 47 ft. As soon as the caissons were firmly bedded in the sand, the wells were filled with concrete and the pier masonry was started at a depth somewhat below low water.

The experience gained in sinking these caissons showed that it is not advisable to splay out the outside walls of a caisson. If this is done, the guiding effect of the surrounding material is largely lost, and a very troublesome condition obtains when on one side the earth is firmer than on the other, thus standing for some time after the other has fallen in, as a consequence of which the caisson is forced out of position.

9-12. Construction with Concrete. The concrete type of open caisson with dredging wells offers a number of advantages over those made of wood or metal. It does away with the uncertainty of future decay or corrosion; its greater specific gravity, compared with wood, makes less weighting necessary in sinking than when timber types are used; its cost will compare favorably with the other forms; and with its concrete filling it forms a monolithic foundation which may be more satisfactory than any combination structure. Where the cellular type is used in which the dredging wells are left unfilled, the concrete is very heavily reinforced and all walls are thoroughly tied together. The one disadvantage of the concrete type is the time element involved in allowing the concrete to harden, but with the advent of quick-hardening cement this objection has been largely eliminated.

The caisson may be entirely of concrete, or it may be a combination of concrete and a structural steel frame. In either type the cutting edge is of steel. Among the notable bridges recently built with all-concrete caissons are the New Orleans bridge (1935) and the Suisun Bay bridge (1930), while among those using a structural-steel framework are the San Francisco-Oakland bridge (1936), the Mid-Hudson bridge (1928), and the Tacoma Narrows bridge (1939). In the first two mentioned the structural-steel framework was on the lower part of the caisson only, whereas in the Tacoma Narrows caissons it extended to the top.

The caissons of the New Orleans bridge were 65 by 102 ft. in plan, and they rest on a stratum of sand 170 ft. below Gulf level and 188 ft. below maximum working stage of the river. These caissons were of the cellular type (Fig. 9-12*a*), all walls being about 4 ft. thick and heavily reinforced with both horizontal and vertical rods near each face. The interior walls were increased to a thickness of about 9 ft. near the top. The sand-island method (Art. 9-14) was used in placing and sinking the caissons. A novel feature of these caissons was the placing of closely spaced vertical 8-in. diameter tubes in both the exterior and interior walls to provide

access to the cutting edges for jetting, drilling, or cutting away obstructions encountered during sinking.

The soil pressure was distributed into the several bearing walls of the caisson by a tremie concrete seal 19 to 21 ft. thick. To keep the load down, the dredging wells were filled only with water above the top of the seal concrete. The maximum soil pressure is only approximately 7 tons per sq. ft., or slightly under 3.8 tons in excess of the natural pressure at foundation level.

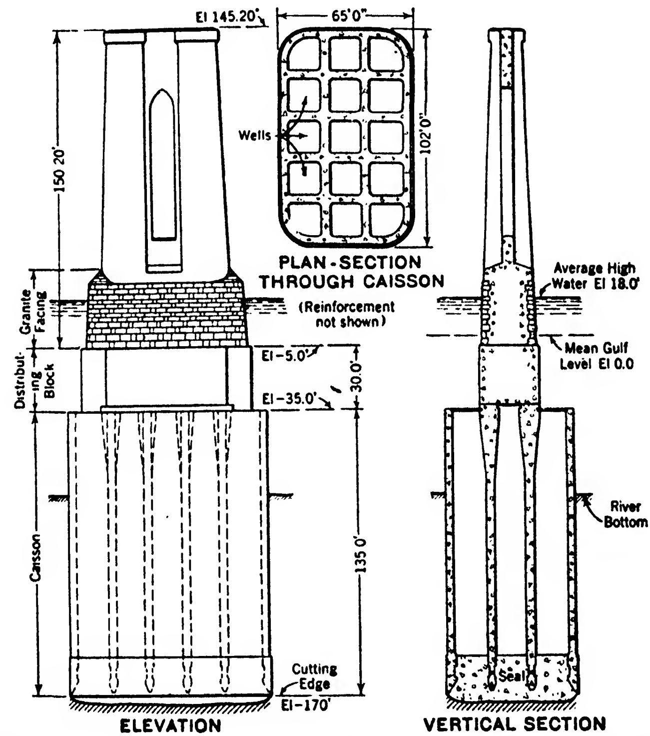


FIG. 9-12a.—Reinforced-concrete Open Caisson with Dredging Wells of the New Orleans Bridge. (*Courtesy of Civil Engineering.*)

The pier proper is carried on a distributing block consisting of a reinforced-concrete slab 30 ft. high set over the central dredging wells. The distributing block and the pier masonry below water level were built within a cofferdam composed of 12- by 12-in. and 8- by 12-in. timbers, operated under a maximum head of 48 ft.

With concrete placed at a maximum depth of 242 ft. below sea level, the caisson for pier 3 of the East Bay of the San Francisco-Oakland bridge holds the world's record for depth of bridge

foundations. The depth of the cutting edge is 228.42 ft., while the average depth of the bottom of the concrete is 235.6 ft. This caisson was 80 by 134½ ft. in plan, six cross walls and three longitudinal walls about 3 ft. thick forming a cellular structure with 28 dredging wells (Fig. 9-12*b*), most of which were about 15½ ft. square. The exterior walls were approximately 4 ft. thick.

The work was started by constructing a base section of structural steel, similar to that shown in Fig. 9-14*c*, heavy cross walls bracing the same and dividing the caisson into wells. Cross walls and outside walls were built hollow and constituted the forms for the first

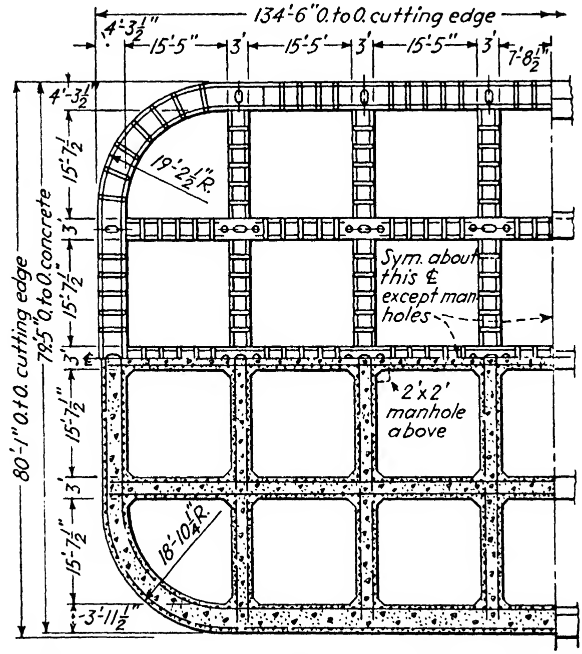


FIG. 9-12*b*.—Caisson for Pier 3 of the East Bay of the San Francisco-Oakland Bridge.

pour of concrete. The outside forms for the reinforced concrete above the structural-steel section for a height of 66 ft. was 4-in. timber, which was left in place. Outside forms above this elevation, as well as all inside forms above the cutting-edge section, were of steel, made up in panels and moved up as the work progressed. As in the case of the New Orleans bridge a concrete seal was placed at the bottom and the wells were left unsealed. The bottom was inspected by divers before the seal was placed. The average rates

of sinking for piers E3, E4, and E5 were, respectively, 1.83, 1.08, and 1.13 ft. per calendar day.

Each 60- by 136-ft. caisson of the Mid-Hudson bridge consisted of a lower 20-ft. section composed of a concrete-filled steel shell braced by a heavy structural-steel frame, the remainder of the height being of reinforced concrete with an outer shell of 4-in. timber. The outer walls were $3\frac{1}{2}$ ft. thick, while the partition walls, $2\frac{1}{2}$ and 3 ft. thick, divided the caissons into 25 cells. A system of steel columns at all wall intersections extended upward and carried 4- by 3-in. horizontal angles at 4-ft. spacing as supports for the outside planking and for the inner wall forms.

9-13. Compressed-air Flotation Caissons. A new type of caisson was developed by the late Daniel E. Moran for the foundations of the piers of the West Channel of the San Francisco-Oakland bridge. These caissons were cellular structures in which vertical steel cylinders, forming the dredging wells, were capped with removable domes to permit air pressure being used. At one of the pier sites rock depth was 224 ft. below water level and at another site the tidal channel was 105 ft. deep to mud line. By proper manipulation of air pressures in selected groups of cylinders the construction risk was reduced through the increase in caisson stability due to a lessened tipping tendency. Another advantage was secured during the landing stage when the caissons changed from floating structures to fixed ones, as it made possible placing the caissons in almost exact position. The three principal steps in the process are illustrated in Fig. 9-13*a*.

The largest caisson was 92 by 197 ft. in plan, sunk 200 ft. below water level. A second caisson, illustrated in Fig. 9-13*b*, $74\frac{1}{2}$ by 127 ft. in plan, had its rock foundation 224 ft. below water level. The cutting-edge sections consisted of box girders in both directions, leaving openings 15 ft. square. For the largest caisson the girder depth was $17\frac{1}{2}$ ft. and for the others $13\frac{1}{2}$ ft. These girders were closed at the bottom so that the section itself was a floating body. Adapter sections connected the square openings between the girders to the 15-ft. diameter steel cylinders. The exterior walls of the caissons above the cutting-edge sections consisted of 10-in. vertical timber sheeting, covered on the outside with creosoted-timber diagonal sheeting, the latter being calked. The walls were braced with horizontal 21-in. H-section waling beams and 8-in. H-section struts, the sizes and spacing being as shown in the illustration.

The cutting-edge sections were assembled on shipways. After launching, and before towing to the site, the adapter sections, the

first lift of the cylinders, the domes, and the first lift of the walls and bracing system were erected and fastened in position. After towing to position, the caissons were held in place against a current of $7\frac{1}{2}$ m.p.h. by a number of anchors placed about 300 ft. from the caisson, this distance being long enough so that the 6-ft. range of tide would not materially affect the tension in the anchor lines.

The details of sinking are well shown by the history of the 92- by 197-ft. caisson. This structure was built to a height of $77\frac{1}{2}$ ft. before being towed to the site. Concrete was placed in the outside and cross walls of the cutting-edge sections to add to the stability

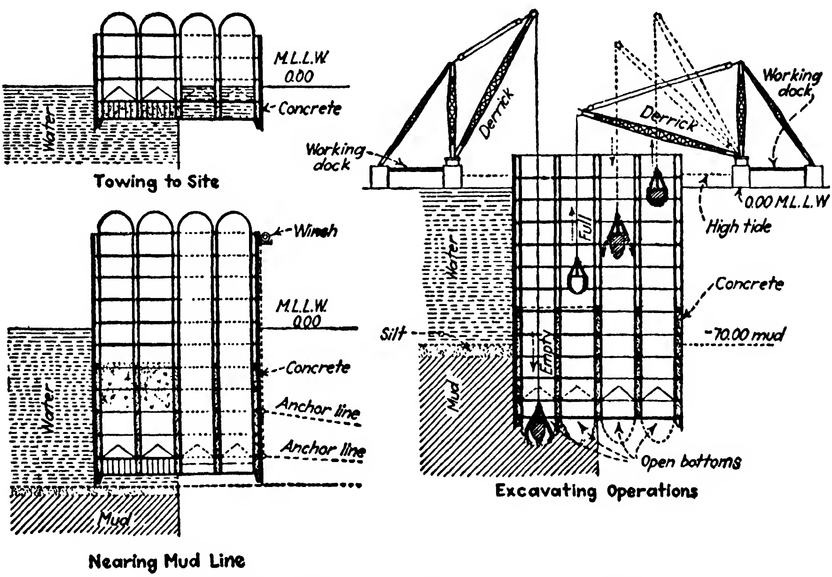


FIG. 9-13a.—Sinking Compressed-air Flotation Caissons.

during towing operations. At this stage it had a draft of 20 ft. and a freeboard (height above water) of about 57 ft. Sinking was effected by adding 5- to 15-ft. layers of concrete in the spaces between well cylinders and between cylinders and outside walls. As concrete was added the air pressure was gradually increased to maintain a minimum draft. With concrete 44 ft. above the cutting edge, the air pressure was 17 lb. and the draft 55 ft. When the structure was sunk to a freeboard of about 15 ft., additional sections of walls and bracing were placed. Air was released in 5 to 7 of the 55 cylinders, domes cut, cylinder extensions welded in place, and redomed at the higher level, this process being repeated until the caisson reached bottom.

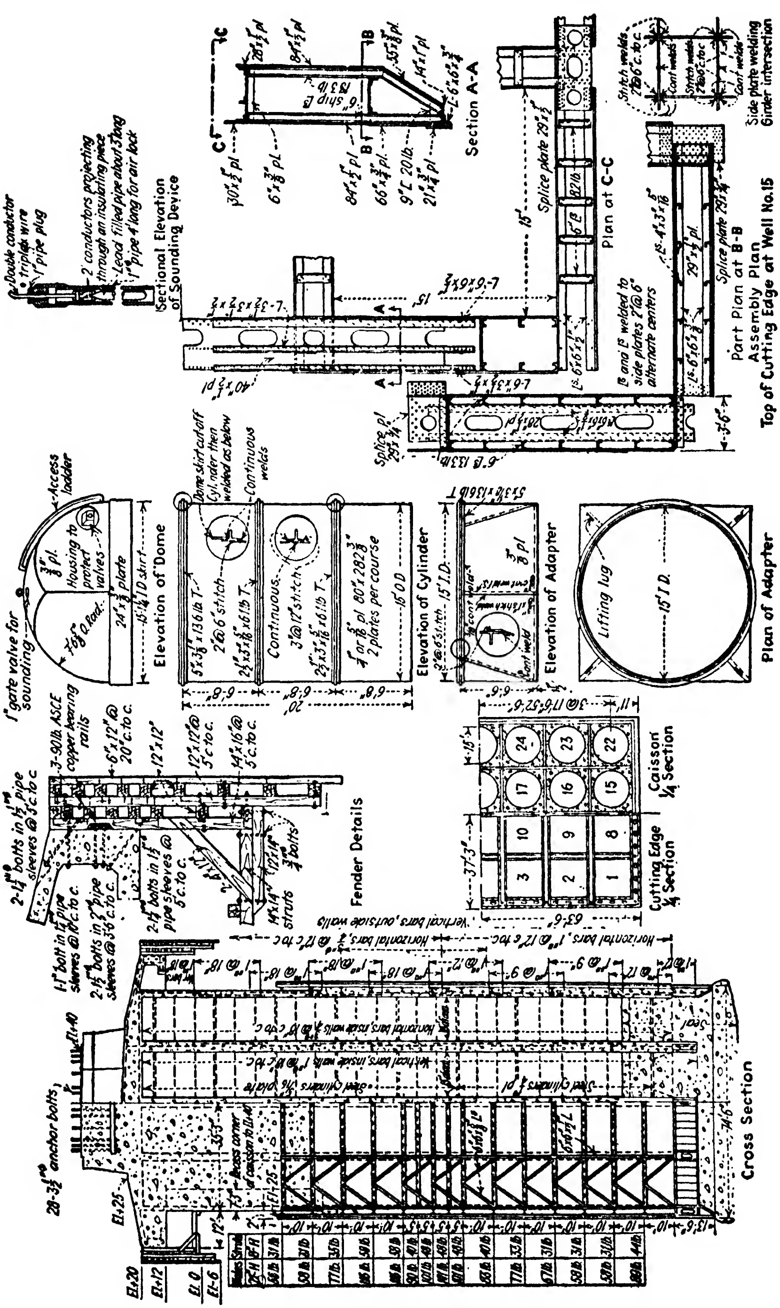


FIG. 9-13b.—Compressed-air Flotation Caisson Used on the San Francisco-Oakland Bridge.

After landing, the sinking process included undercutting the cutting edges and cross walls by removing mud and clay through the dredging wells when the domes were removed. When stiffer material was encountered, air pressure was released and the domes removed entirely.

On the completion of the sinking, the bottom was thoroughly cleaned to bedrock. Water-jets, working under pressures up to 350 lb. per sq. in. were used to remove the material along the cutting edges and under the girders, while suction pumps were used to remove fine sand and mud on the bottom. Final digging with toothless dredging buckets removed loose-rock fragments. Frequent diver inspections were made to facilitate cleaning, these inspections being made to depths of 220 ft. Special bottom-dump buckets were used for placing the concrete seal, this seal extending 34 ft. above the cutting edge. The last stage involved building the piers above water level. The cylinders were left unfilled.

9-14. Building and Placing Open Caissons. The building and placing of box and single-wall open caissons where only moderate depths are involved, require no special ingenuity, and the methods commonly used are described in the foregoing articles. However, in the case of large caissons sunk to great depths, especially where the water is deep and the current swift, great skill is required. In general, the methods are similar to those used for pneumatic caissons, described in Art. 10-11.

The sand-island method, which is patented, consists in creating an artificial island to furnish lateral stability and frictional resistance, particularly needed for concrete caissons placed in swift streams where the soil penetrated is silt or other soft soil. This method was first used in connection with the building of eight piers of the Martinez-Benicia bridge across Suisun Bay in 1930. Here borings showed a maximum water depth of 55 ft., mud to a depth of about 90 ft. below low water, and rock at a maximum depth of 143 ft., averaging 116 ft. below low water. The current attained a velocity of 5 m.p.h. A steel cylinder, 81 ft. in diameter, was sunk from an octagonal dock built around the pier site, as shown in Fig. 9-14*a*. The cylinder was brought to the site as an assembly of ½-in. steel plates 10 ft. wide and of a length equal to one-eighth of the circumference. This was handled by eight winches. The cylinder was reinforced with angles and plates at various points and also by horizontal truss bracing placed halfway up each of the 10-ft. sections, this bracing being so arranged as to leave an unobstructed rectangle 69 by 50 ft. in place in the center. The cylinder

was allowed to sink freely of its own weight, and, when it came to rest, the mud was dredged from the inside and the shell back-filled with sand nearly to the top. On the completion of the caisson sinking, most of the cylinder was salvaged.

The second application of the sand-island method was in connection with building the New Orleans bridge (Art. 9-12), the diameter of the largest cylinder here being 122 ft. The material underlying the bed of the Mississippi River at New Orleans consists of silt, sand, and clay to unknown depths. As the river bed is easily eroded, prior to placing the cylinders loosely woven willow mat-

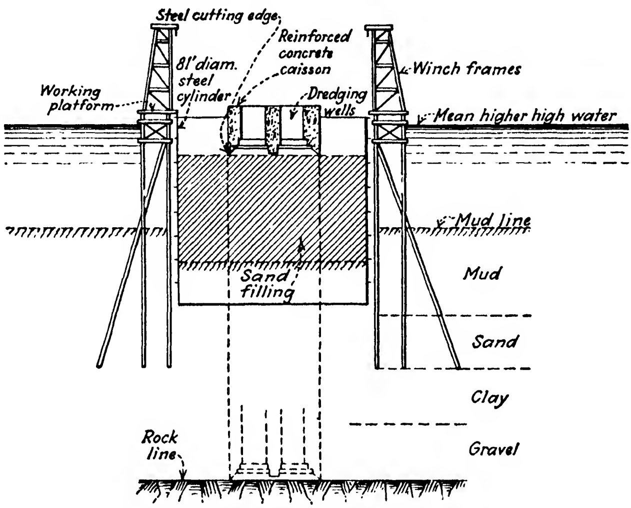


FIG. 9-14a.—Illustrating Sand-island Method of Sinking Open Caissons.

tresses 250 ft. wide and varying in length up to 450 ft. were sunk with riprap to protect the sites from scour.

In placing the steel cylinder caissons of the new Atchafalaya River bridge (Art. 9-5) at Morgan City, La., steel sheet-pile coffer-dams were used to create still water in the tidal stream, the water being about 53 ft. deep. The cutting-edge sections of the caissons, 30 ft. high, were fabricated in Pennsylvania and shipped by barge to the site.

Figure 9-14*b* illustrates the method used in placing and guiding the caissons of the Carquinez Strait bridge (Art. 9-10), where the water depth, including scour, was 100 ft., with a current velocity of 5 m.p.h. The anchorage system consisted of a set of six structural-steel columns 120 ft. long, driven vertically into the mud. The tops of the columns were framed together, as shown in the illustration,

and braced by $1\frac{7}{8}$-in. cables attached to 16,000-lb. ship anchors. Three sides of an anchorage frame were first completed, after which the caisson, which had been erected on shore to a height of 19 tiers, was towed to the site and placed in position. The rest of the framing was then placed. In building a suspension bridge at Tacoma, Washington, where the water depth ranges from 80 to 180 ft. and tidal currents are frequently 6 to 10 m.p.h., each caisson was sur-

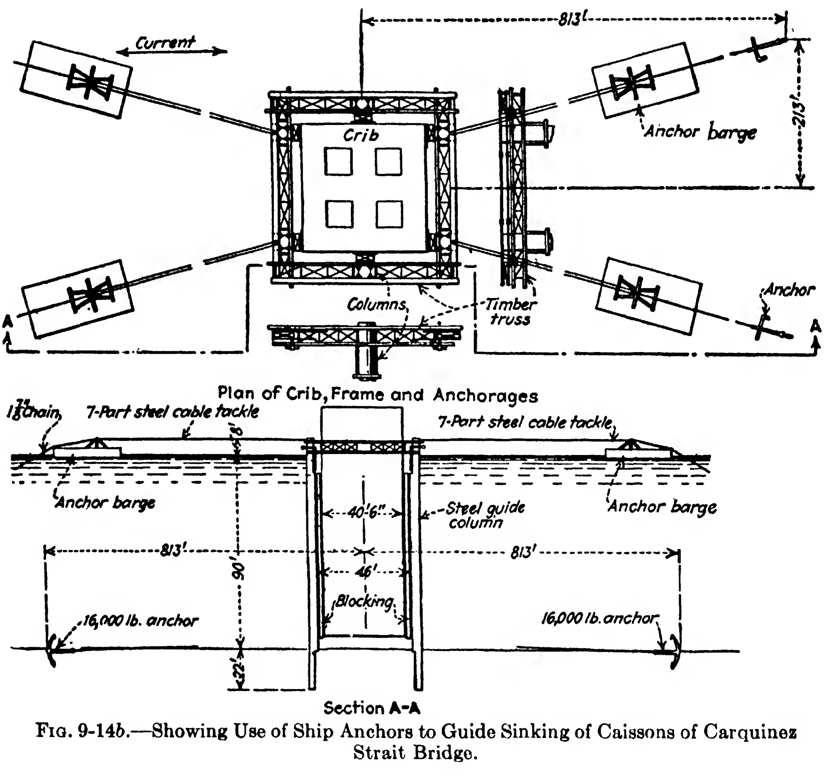


Fig. 9-14*b*.—Showing Use of Ship Anchors to Guide Sinking of Caissons of Carquinez Strait Bridge.

rounded by at least 24 concrete anchors placed on the bottom in the circumference of a circle 924 ft. in diameter. These anchors had dimensions of 12 by 12 by $51\frac{1}{2}$ ft. and weighed 600 tons.

In placing and sinking caissons in deep water, the required buoyancy becomes an important problem. In making studies for the Carquinez Strait bridge, computations indicated that 1 ft. of concrete in the crib chambers would sink the crib approximately 1.6 ft. and that 1 ft. of timber-crib construction would cause a further sinking of 0.3 ft. This necessitated extending the crib and

dredging-well walls and crib bracing up well in advance of concrete placing, and designing them for a maximum head of 45 ft.

Initial buoyancy may also be obtained by the use of temporary false bottoms in the dredging wells. This eliminates the necessity of designing the crib for high hydrostatic heads and is the only practical method where concrete caissons are used. Figure 9-14c shows the construction of the false bottoms used for the East Bay

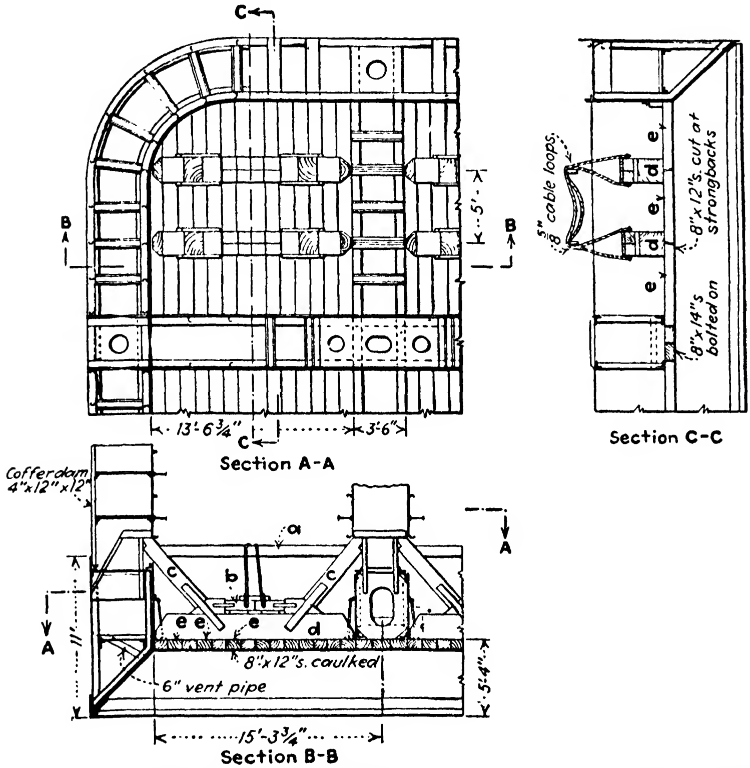


FIG. 9-14c.—Showing False Bottom of Caisson Used on the San Francisco-Oakland Bridge.

caissons of the San Francisco-Oakland bridge. The success of this method depends largely on the ease with which these false bottoms can be removed. In this case a steel cable was attached to the members lettered *a*, *b*, *c*, *d*, and *e*, enough slack being allowed between the several points of attachment so that each member could break out and come free before the pull was applied to the next member. Figure 9-14*d* shows the false bottom of the New London bridge open caisson described in Art. 9-10. The steel cutting edges were assembled on the false bottom resting on ways, and the timber

work was carried up about eight courses. Concrete was placed in the cutting edges, and, after hardening, the caisson was launched, towed to the site, and further built up as high as possible and still permit the removal of the bottom. The dredging wells were then filled with water to river level by pulling out the calking stick. Enough gravel was dumped into the dredging wells to overcome the buoyancy of the false bottom. Difficulties involved in taking out the false bottom of one of the caissons of the Mid-Hudson bridge was the indirect cause of the tipping of the structure, which required almost a year to right.

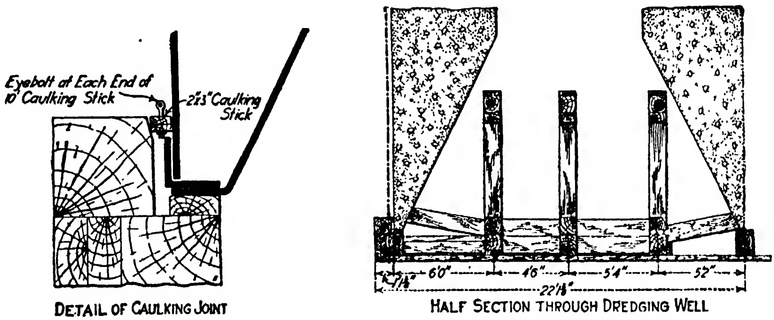


FIG. 9-14*d*.—False Bottom of New London Bridge Caisson.

9-15. Sinking Open Caissons. Open caissons for bridges may be sunk by a number of methods according to the type and size of the caisson and the material penetrated. In general one or more of the following agencies are involved: (*a*) removing the underlying material, (*b*) weighting the caisson, (*c*) reducing side friction by using water-jets, (*d*) driving, (*e*) pulling down, and (*f*) boring. In sinking caissons for buildings the jacking method (Art. 9-6) has been used.

For the small caisson to be sunk only a few feet, as practiced by the natives of countries in the Far East, excavating is done by buckets carried down, filled, and brought up by divers. The modern method consists of dredging or pumping out the spoil. Dredging is usually done with an orange-peel or clamshell bucket. This is most effectively done where the dredging wells occupy a large proportion of the plan section of the caisson; in other words, where the exterior and interior walls are thin. Where a layer of stiff clay is met, it may be broken up by sending divers down to blast it to pieces, or it may sometimes be broken up by dropping down steel rails in a vertical position. These sink into the clay and tear it up in tipping over.

The mud and sand pump, the principle of which is described in Art. 10-13, is used where the material is largely silt or sand. Figure 9-15*a* shows a type of ejector used in France. It consists of a 10-in. pipe with a toothed bottom to penetrate the sand, inside of which is a gas pipe connected with compressed air. The gas pipe is perforated at the bottom with small holes, through which the escaping air churns up the sand and aerates the water in the ejector pipe, causing it to rise and flow out of an outlet at the top.

The most economical way of weighting caissons is to make use of the permanent filling, and for this reason, where the size of the caisson makes it possible, a double wall should always be used. Temporary weighting, as with rails laid on top, is always expensive on account of the time and labor involved, and moreover it interferes with dredging operations.

The water-jet (Art. 4-7) is always a useful adjunct in caisson-sinking operations. As explained in Art. 9-10, the outside jets serve to reduce side friction, while the inside ones cut the material away from the cutting edges and move it toward the center of the wells. Movable water-jets are frequently used by divers, particularly in the final cleaning off of the rock foundation.

It is possible to drive caissons only when they are small. Pulling the caisson down may sometimes be employed to advantage, if it is possible to drive piles around the outside and attach tackle to them and to the caisson.

The latest method is to bore caissons into position as explained in Art. 9-7. This method is applicable only to small metal-cylinder caissons and is more widely used in building construction than for bridges.

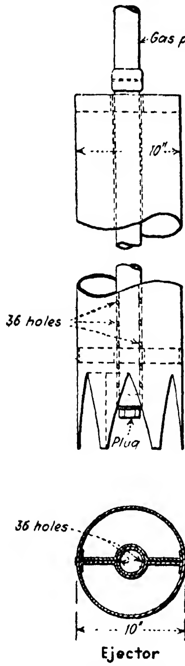


Fig. 9-15*a*.—Sand and Mud Ejector.

In sinking deep caissons through soft soil it is often hard to keep the caissons from tilting. To reduce this tendency, experience has shown that (*a*) the weight of the caisson should be kept low and its center of gravity as near the cutting edge as possible; (*b*) where soil conditions are uniform, dredging and the removal of false bottoms should be kept symmetrical; (*c*) dredging operations should maintain rim bearing and relieve internal wall bearing; and (*d*) opposite ends should be sunk in succession, as tipping is most probable along the long axis of the caisson.

CHAPTER X

PNEUMATIC CAISSONS FOR BRIDGES

10-1. The Pneumatic Process. A pneumatic caisson may be defined as a structure, open at the bottom and closed at the top—in other words, an inverted box—in which compressed air is utilized to keep the water and mud from coming into the box, and which forms an integral part of the foundation.

The caisson, which is usually not over 6 ft. high in the working chamber, is surmounted by a crib and cofferdam, the former, with the exception of one or more vertical wells, called "shafts," being filled with concrete as the caisson sinks. This concreting, together with the excavating done in the working chamber, as the interior of the caisson is called, effects the sinking of the latter.

The working chamber must be practically air- and watertight, and yet there must be an opening for men to enter and leave the chamber, as well as an inlet and outlet for materials. These openings are provided by vertical shafts and air locks. The shafts, which extend from the roof of the caisson to a point well above the top of the crib and the level of the water outside, are usually of a circular or oval section and from $2\frac{1}{2}$ to 4 ft. in maximum diameter. In the shafts, usually at the top, are placed the air locks, they being airtight chambers, often simply a part of the shaft itself, fitted with two doors, one of which leads to the working chamber and the other to the open air.

The most pronounced advantages of the pneumatic-caisson as compared with the open-caisson process lie in the fact that the engineer has more control over the work, having a better opportunity to sink the caisson vertically and to remove large boulders, sunken logs, etc., from under the cutting edge; the foundation bed can be properly prepared and personally inspected; and, lastly, the concrete filling of the working chamber is deposited in air, thus giving a superior foundation. Another point, which is sometimes of great importance in placing foundations for buildings, is that the soil about the caisson is not so liable to be disturbed when the pneumatic process is used. The one disadvantage of this process is that the men have to work under an air pressure which is sufficient

to balance the pressure of the surrounding water in addition to atmospheric pressure, or practically the full hydrostatic head from the cutting edge to the water surface.

For depths from about 40 to 110 ft. this type of caisson is extensively employed. For depths less than 40 ft. the cofferdam process is usually, but not always, a more economical method of placing the foundation, whereas for depths greater than about 110 ft., corresponding to a pressure of over three atmospheres above the normal, the open-caisson method must be employed, since men cannot work advantageously under such high pressures. Probably the minimum depth for which a pneumatic caisson was used was for an elevator pit near a high building in New York in 1909, where the depth below water level was only 6 ft. The presence of quicksand made open excavation methods too risky.

The maximum depth (1941) below water surface for bridge pneumatic-caisson construction is 125 ft., this depth having been reached in placing the caisson for pier 2 of the bridge over the Kennebec River at Bath, Maine. The pressure used was over 50 lb. In placing the caisson of the Raritan River highway bridge at Perth Amboy, a maximum depth of 115 ft. was reached; however, the maximum pressure used was much less than for some other bridges. A depth of 115 ft. was also reached in sinking the caissons of the Grey St. bridge in Brisbane, Australia.

Among other notable examples of deep immersions are the Metropolis bridge, 113.2 ft. and 51-lb. air pressure; the Vicksburg bridge, 112.3 ft. and 53-lb. air pressure, the maximum ever used in this country; the St. Louis Municipal bridge, 112 ft. and 50-lb. air pressure; the Boulak bridge over the Nile at Cairo, 111.5 ft.; the Lexington bridge over the Missouri, 110 ft. and 52-lb. air pressure; the St. Louis arch bridge, 109.7 ft.; the Williamsburg bridge (New York), 107.5 ft.; and the old Memphis bridge, 106.4 ft.

The first use of compressed air was made by Triger, a French engineer, in sinking a shaft in 1839. This method was used for placing bridge pier foundations in Europe in 1851 and in the United States in 1852, the latter being for the foundations of bridges over the Pedee and Santee rivers. Here the caissons consisted of cast-iron cylinders, called "pneumatic piers," which formed both the working chambers and sections of the piers.

The St. Louis arch bridge was the first in this country to be founded on large pneumatic caissons, its east abutment caisson, which had a maximum immersion of 109 ft. 8½ in., being sunk in 1870. The second bridge in this country to be founded on large

pneumatic caissons was the great Brooklyn suspension bridge, which, in its New York tower caisson, sunk in 1871, has the largest pneumatic caisson ever placed for a bridge foundation. It was 102 by 172 ft. in plan and was sunk to a depth of 78 ft. below high-water level.

Following is a list of the largest pneumatic caissons built to date (1941):

Date	Bridge	Dimensions	Area, square feet
1871	New York pier, Brooklyn bridge	102 × 172	17,544
1898	Alexander III bridge, Paris	110 × 145	15,850
1901	Manhattan bridge, New York	78 × 144	11,232
1922	Delaware River bridge	70 × 143	10,610
1910	Quebec bridge	55½ × 180½	10,018
1914	Metropolis bridge	60½ × 110½	6,655
1869	East abutment, St. Louis arch bridge	72½ × 82 (hex.)	6,000

The pneumatic-caisson process has been widely used in America and on the European continent. American engineers early developed the wooden caisson to a high state of perfection, but the tendency now is toward the use of steel or concrete. Steel caissons have been widely used in Europe.

10-2. Roof Construction of Timber Caissons. In the early pneumatic timber caissons the roof construction was extremely heavy. For example, the roof of the 102- by 172-ft. caisson for the New York tower of the Brooklyn bridge, built in 1871, was composed of a solid mass of squared timbers, 22 ft. thick, all timbers being 12 by 12 in. in section and thoroughly drift-bolted together. The roof of the 31- by 79-ft. rectangular caisson of the Havre de Grace bridge built in 1884 was composed of eight thicknesses of 12- by 12-in. timbers, the courses alternating in direction, some running longitudinally, others transversely, and still others diagonally. The lower surface was sheathed with 3- by 12-in. planks.

When concrete superseded stonemasonry as a filling for the crib, a considerable decrease in roof thickness was made possible by the strength of the concrete. A more generous use of bulkheads and the arrangement of the bracing above and below the deck to act as trusses also aided in securing a thinner roof. By reinforcing the concrete in the cribs the timber roof may be almost entirely dispensed with.

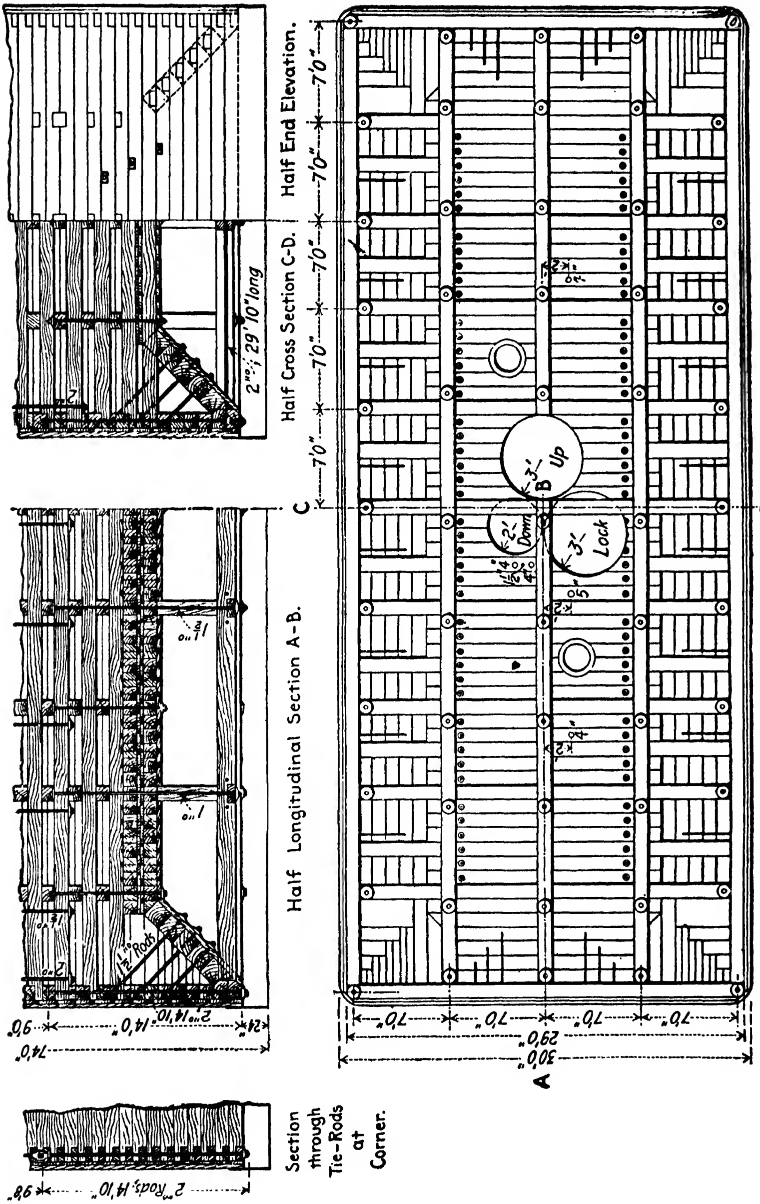


FIG. 10-2a.—Pneumatic Caisson for Pier 4 of Bellefontaine Bridge. Designed in 1892.

Typical of timber roofs of moderate thickness is that of pier 4 of the Bellefontaine bridge, built in 1892. As shown in Fig. 10-2*a*, it consisted of two courses of large-size timbers, between which were placed two courses of sheathing laid diagonally. The lower side of the roof was also lined with sheathing. Tongue-and-groove sheathing, well calked, served to make the structure airtight. This type of roof, which is typical of many designs made by George S. Morison, was considerably stiffened by connecting the roof to the bracing timbers of the crib above by means of tie rods.

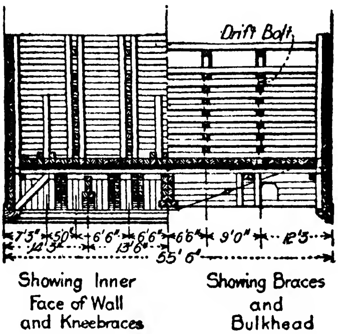


FIG. 10-2*b*.—Quebec Bridge Caisson.

In the 55.5- by 180.5-ft. caisson for the south main pier of the second Quebec bridge the roof consisted of one solid course of longitudinal and one solid course of transverse timbers, separated by two crossed courses of diagonal 3-in. tongue-and-grooved planks. Eight transverse bulkheads extending the full width of the caisson, one longitudinal bulkhead at the center extending the full length of the caisson, and four longitudinal ones extending only from the

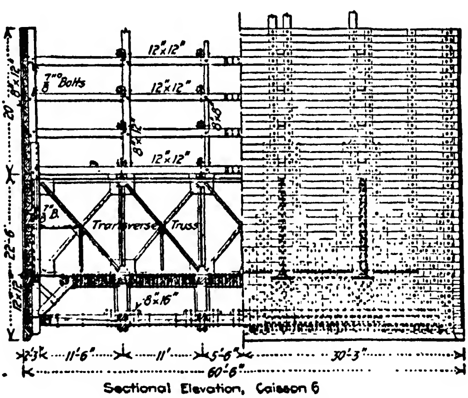


FIG. 10-2*c*.—Pneumatic Caisson for Metropolis Bridge, Metropolis, Ill.

ends to the first transverse bulkhead served to stiffen the roof (see Fig. 10-2*b*).

Figure 10-2*c* shows the 60½- by 110½-ft. caisson of the Metropolis bridge at Metropolis, Ill., where only a single thickness of large timber was used for the roof. The concrete above was heavily reinforced, steel trusses taking the weight of the concrete until it

had hardened. Much the same type of roof was used for the caissons of the Municipal bridge at St. Louis, except that in this case the bracing in the crib was trussed (Fig. 10-3*a*) to form wooden trusses.

10-3. Sides of Working Chamber. The sides of the caisson should be made strong and rigid enough to take not only the direct vertical loads but also to safely withstand sudden lateral thrusts, eccentric loads due to unequal sinking on opposite sides, etc. For the prevention of leakage of air outward and of water inward all joints must be thoroughly calked. The required thickness of walls will depend somewhat on the clear height of the working chamber, as well as on the kind of soil through which the caisson is to be sunk. The height is usually about 6 ft.

The sides should be vertical on the outside, for to batter the same to reduce friction is to invite trouble. Such a design makes it more difficult to sink the caisson plumb and is apt to increase rather than to decrease friction by allowing boulders to roll into the open space.

Practically all working-chamber sides are constructed of one of the two following forms: (*a*) a double wall with a V-shape section

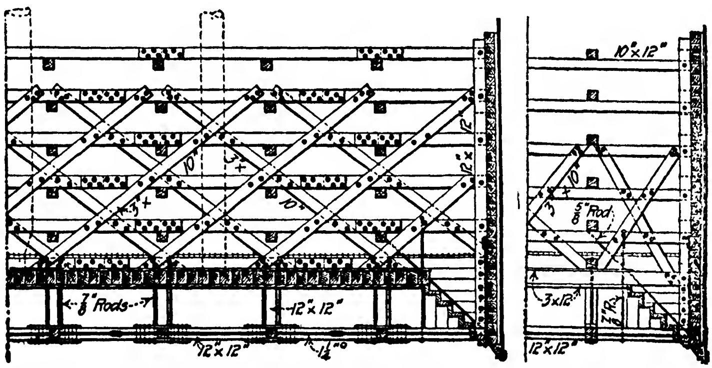


FIG. 10-3*a*.—Half Longitudinal and Half Transverse Sections, Municipal Bridge.

and (*b*) a single wall with a rectangular section. The former is more rigid, whereas the use of the latter permits excavation under the cutting edge to be more easily made.

The first type is illustrated in Fig. 10-2*a*. Here the outer wall was made of 12- by 12-in. timbers, sheathed on the outside with two layers of planking, the outer one running vertically and the inner one diagonally. The inner wall consisted of a single thickness of 17- by 17-in. timber sheathed with 4-in. planking and tied to the

outer walls with rods. The space between the walls was filled with concrete.

The inner wall may be stepped as was done in the caissons of the St. Louis Municipal bridge as shown in Fig. 10-3*a*. The outside wall consisted of a single layer of 10- by 12-in. timbers, sheathed on the outside with two courses of planking, one, 3- by 12-in., running diagonally and the outside one, 2- by 12-in., running vertically. The inner wall was formed of 4- by 12-in. planks, stepped and supported every 10 ft. on vertical struts. The small size of material used in this wall was made possible by reinforcing the concrete in the space between the walls. Stepping the walls made it possible to count on its horizontal projection to take load when the caisson was finished and filled with concrete.

The single-wall type of side wall is illustrated in Fig. 10-2*b*. It is composed of two thicknesses of large timbers, one placed horizontally and one vertically, with two layers of sheathing on the outside. In the caissons used on the Broadway Bridge in Portland, Ore., the walls were composed of a double thickness of horizontal 12- by 12-in. timbers, separated by vertical timbers of the same size, some of which extended up beyond the caisson to form a part of the crib. Sheathing was used on both outside and inside faces.

The design of the walls of the Delaware River bridge caissons, described in Art. 10-6, represents a considerable advance in caisson-design practice as shown by the following table, taken from an article by C. E. Chase, published in the *Journal of the Franklin Institute*, November, 1923. This table gives a comparison of the strength of the working-chamber walls of four caissons.

Caisson	Outward thrust per foot, pounds	Inward thrust per foot, pounds
Second Quebec bridge	2,000	26,000
Manhattan bridge	5,000	36,000
Metropolis bridge	7,000	87,000
Delaware River bridge	55,000	70,000

10-4. Cutting Edges and Caisson Bracing. The cutting edge, as the part of the caisson that rests on the soil is called, should be designed (*a*) to stand the strains and abrasive action of sinking, (*b*) to allow the caisson to sink without excavating under the cutting edge, (*c*) to prevent sudden sinking on reaching a soft stratum, and (*d*) to prevent the escape of air. To fulfill requirement *a*, the

cutting edge is usually made of some tough and strong wood, such as elm, or else is shod with a metal plate or piece of tough wood. Requirements *b* and *c* are conflicting; for *b* a true knife-edge is the ideal form; and for *c* a considerable breadth of bearing is desirable. As constructed, the width will vary from about 4 to 18 in. To meet requirement *d*, a vertical plate extending about 6 in. below the cutting edge is often used. Where the soil is dense, this plate may not be needed.

Figure 10-2*b* illustrates the use of a special timber wearing plank on the cutting edge. It was 6 by 12 in. in section, the main timber forming the cutting edge being 30 by 30 in. in section.

One form of metal cutting edge, illustrated in Fig. 10-4*a*, consists of a horizontal and vertical plate, the latter being stiffened at intervals and fastened to the horizontal plate by steel diaphragms. As used for the caisson of Fig. 10-2*a*, the horizontal plate extended under, and was fastened to, both the lower surface of the bottom timbers and the lower edge of the outside sheathing, while the ver-

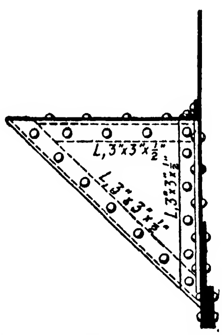
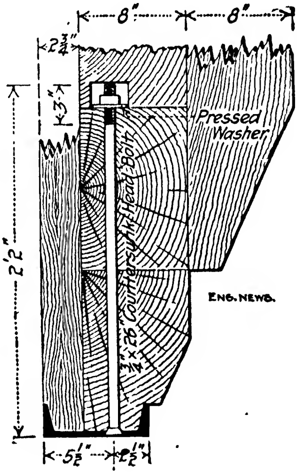


FIG. 10-4*a*.—Details of Cutting Edge.



Detail of Cutting Edge. (Enlarged)

FIG. 10-4*b*.

tical plate was fastened to the same outside sheathing. Another form of metal cutting edge has a vertical plate on the outside and a horizontal angle with its vertical leg down and fastened by rivets to the vertical plate and with its horizontal leg fastened to the lower surface of the bottom timber. Figure 10-4*b* illustrates a metal cutting edge consisting of a channel iron.

Various types of bracing systems are used for pneumatic caissons. One type may be of struts and ties near the bottom, running horizontally the length and breadth of the caisson or it may be in the form of bulkheads or trusses. The latter two usually serve the

added purpose of supporting the roof. The strut-and-tie type is illustrated in Figs. 10-2*a*, 10-2*c*, and 10-3*a*, rods being placed on both sides of the struts. The struts at their intersections are braced with vertical timbers and pairs of vertical rods extending to the deck of the caisson.

The use of bulkhead bracing is well illustrated in Fig. 10-2*b*. The main longitudinal bulkhead was 24 in. thick and the transverse ones were 12 in. thick, except for the lower course, which in each case, was 12 in. thicker. All extended from the ceiling to about the top of the cutting edge. Knee braces were also used to stiffen the walls of the caisson.

10-5. Crib and Cofferdam Construction. The crib is often considered as a part of the caisson, but confusion will be avoided by designating that part of the structure above the caisson roof as the crib. A certain height of crib is often built as an integral part of the caisson to facilitate floating the structure into place. The purpose of the crib is twofold, (*a*) it serves as a form for the concrete and (*b*) it serves to keep out the water. If the masonry or concrete work is kept sufficiently in advance of the sinking, the crib may sometimes be dispensed with, but this is seldom done because it brings too much weight on the caisson. The crib is an integral and permanent part of the foundation and usually its walls are a continuation of the walls of the caisson, perhaps slightly modified. The crib is thoroughly braced with longitudinal and transverse timbers left permanently in place.

Although it is customary to fill the crib with concrete, under some circumstances this may not be done. In the foundation for pier 2 of the old Memphis bridge, where the nature of the soil made it necessary to keep the load on the foundation at a minimum, the pockets near the walls of the crib were left empty, while for about 15 ft. down from the top of the crib a solid-timber grillage was used, thus decreasing the load very considerably.

As shown in Fig. 10-2*b*, the crib for the south main pier of the second Quebec bridge had a wall made of a single thickness of horizontal 12- by 12-in. timbers, braced by inside vertical 12- by 12-in. timbers, the latter being extensions of certain of the vertical timbers forming the sides of the caisson. The outside sheathing was the same as was used for the caisson. The walls were braced with horizontal longitudinal and transverse struts 24 in. apart vertically, dividing the crib into 90 pockets approximately 10 ft. square. For other examples of typical crib construction see Figs. 10-2*a*, 10-2*c*, and 10-3*a*.

Both durability and appearance require that no parts of the crib extend above low-water level, and, moreover, to keep current obstruction a minimum, the crib should generally be stopped and the pier begun at a considerable distance below low water. In some cases, where the current has a high velocity, the pier is started at or below river-bed elevation, or the upper part of the crib may be built with pointed ends. Unless conditions are such that the pier construction can be kept well above water level, as when construction is carried on at low-water stage or when friction and resistance to sinking are large, a cofferdam must be used. Even under these conditions engineers often prefer not to start pier construction until the caisson sinking has been completed because of the uncertainty regarding lateral movement.

The cofferdam walls are usually made of lighter construction than those of the cribs, but they are always thoroughly calked and braced by struts running the length and breadth of the structure. As the pier is built up, these braces are removed and the walls braced against the pier. On completion of the pier the cofferdam is removed.

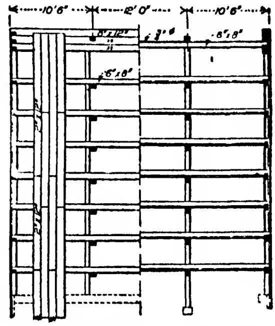


FIG. 10-5*a*.—Framing of Cofferdam, Municipal Bridge.

Figure 10-5*a* illustrates the cofferdam used for one of the piers of the St. Louis Municipal bridge. The cofferdam, which was 33 ft. 7½ in. high, consisted of a frame of horizontal 6- by 8-in. and vertical 6- by 6-in. timbers, sheathed with 2- by 12-in. planks. It was braced with 6- by 8-in. struts, 4 ft. apart vertically, and in rows about 10 ft. apart horizontally.

10-6. Pneumatic Caissons of Metal. The all-metal caisson, of either the open or pneumatic type, has not been widely used in this country, except in the case of cylinder caissons. Abroad metal has been used very extensively. However, the river piers of the St. Louis arch bridge, the first structure in this country founded on large pneumatic caissons (1869), rest on metal caissons. At that time there was no precedent to guide the engineers in the use of timber, whereas metal had already been used in Europe.

The caisson for the east pier, which was hexagonal in shape, with over-all dimensions of 60 by 82 ft., had walls of wrought-iron plates ⅜ in. thick, braced with iron brackets extending from the bottom to the top and spaced 2½ ft. apart. The roof was formed of ½-in. iron plates riveted to the bottom flanges of 13 parallel iron

girders, spaced $5\frac{1}{2}$ ft. apart. The roof was also supported by two heavy bulkheads of oak timber in the air chamber. These strong supports for the roof were necessary because it had to take the entire weight of a 100-ft. height of stonemasonry. The walls of the caisson extended above the roof to form an inclosure in which the stonemasonry was laid. For some distance up the masonry covered the entire cross section of the crib, but above this it was

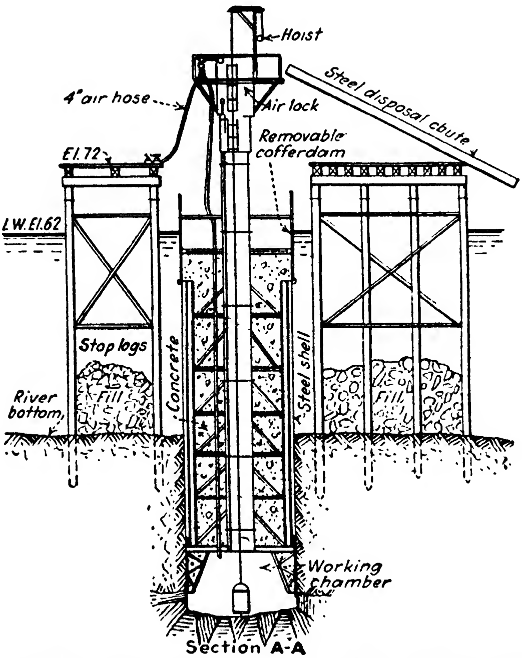


FIG. 10-6a.—Sinking All-steel Caisson of the Caughnawaga Bridge over the St. Lawrence River.

stepped off, the space between the iron envelope and the masonry being braced with timbers and filled with sand.

The metal pneumatic caissons of the Alexander III bridge in Paris, built in 1897, are among the largest ever used (see Art. 10-1). Four transverse girders extended through the working chamber, dividing the chamber into five subchambers, their bottoms forming cutting edges. On their upper flanges these girders supported 27 longitudinal latticed girders. The working chamber had a roof of steel plates 0.2 in. thick which was fastened to the lower flanges of the longitudinal girders and to the upper flanges of the transverse girders. At the sides these plates followed down the inclined end posts of the transverse girders and at the ends followed the knee braces down to the cutting edge to give sloping inside walls on all

four sides. Between these inclined plates and the outer vertical walls was a triangular space filled with concrete. The outside wall plates and the transverse girders were all stiffened with knee braces extending from the cutting edge to the longitudinal girders.

The use of steel caissons permits of more free and unobstructed working room than when timber is used. There is also less leakage of air. However, steel construction is economical only where shipbuilding yards are close at hand.

Figure 10-6*a* illustrates a type of all-steel caisson as used in the construction of the Caughnawaga bridge over the St. Lawrence River 12 miles upstream from Montreal. All steel connections above the cutting edges were welded. The cutting edges were very heavy, having a total thickness of $1\frac{3}{4}$ in. The caissons varied from 14 to 20 ft. in width and were from 60 to 74 ft. long. They were founded on bedrock, the maximum depth below water being 62 ft.

Figure 10-6*b* illustrates the steel caissons used by the New Jersey Highway Department on a bridge over the Raritan River at Perth Amboy, the maximum size of which was $22\frac{1}{2}$ by $61\frac{1}{2}$ ft. The caissons were shop-fabricated to a height of $9\frac{1}{2}$ ft. and, after being placed in position for sinking, were built up by the addition of vertical 6-in. 12-lb. I-beams, spaced 4 ft. apart and covered with 3- by 12-in. planking. These beams were supported against lateral pressure by rings of 10- by 12-in.

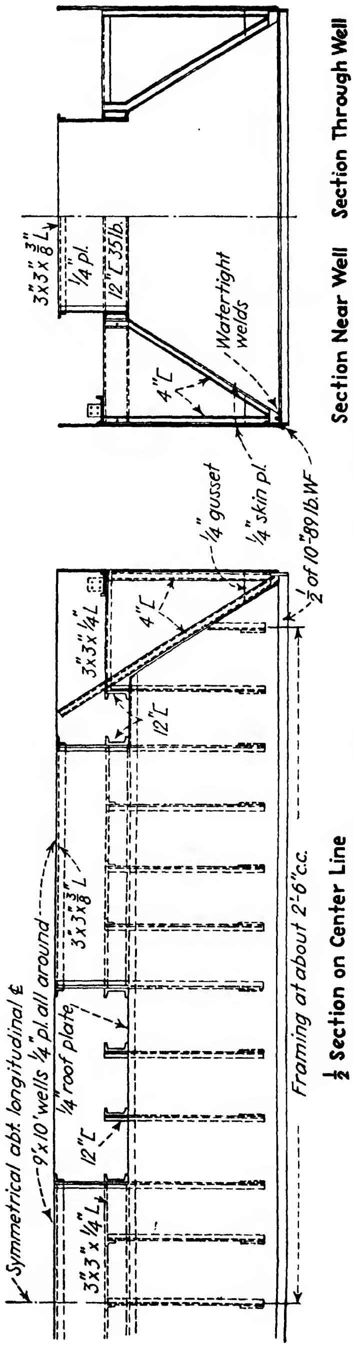


FIG. 10-6*b*.—Steel Caisson for Bridge over the Raritan River at Perth Amboy, N. J.

timbers at 5-ft. spacing, as shown in Fig. 10-6c, these rings being removed as the concrete filling was placed. Three dredging wells were formed in each caisson by steel pipe.

These caissons were sunk by open dredging until hard material was reached. The wells were then decked over, usually with a poured concrete platform, and a 3-ft. shaft for a man lock and two 4-ft. shafts for muck locks were set. Air pressure was applied and

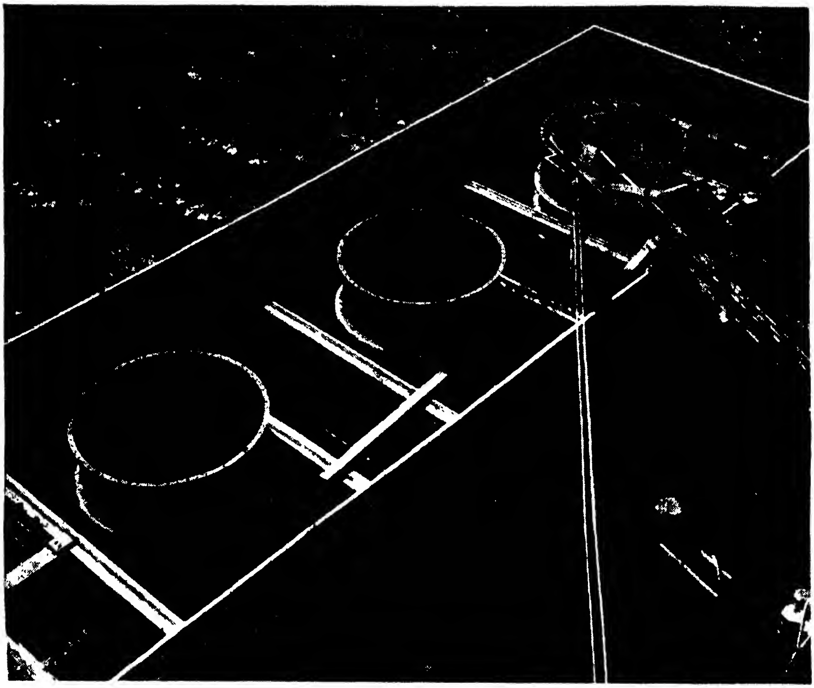


FIG. 10-6c.—Raritan River Bridge Caisson Showing Wood Sheathing Used on Crib. (*Courtesy of New Jersey State Highway Department.*)

the remainder of sinking effected by the pneumatic method. The maximum depth was 83 ft. and the maximum air pressure 34 lb.

During recent years a number of bridges have been built using pneumatic caissons with steel working chambers and timber cribbing. Typical of this combination type is the 24- by 97-ft. caisson of the Newark Bay bridge of the Central Railroad of New Jersey shown in Fig. 10-6*d*. The working chamber, 7 ft. 4 in. high, had V-shaped side walls composed of structural steel frames supporting steel plates. The cutting edge was composed of an angle and plates. The roof consisted of steel plates fastened to the lower flange of channel joists running lengthwise of the caisson and connected to

the lower chord of a series of transverse trusses $9\frac{1}{2}$ ft. deep. The sidewalls of the crib started 8 ft. $9\frac{3}{4}$ in. above the cutting edge and consisted of 10- by 12-in. timbers laid on edge and sheathed on the

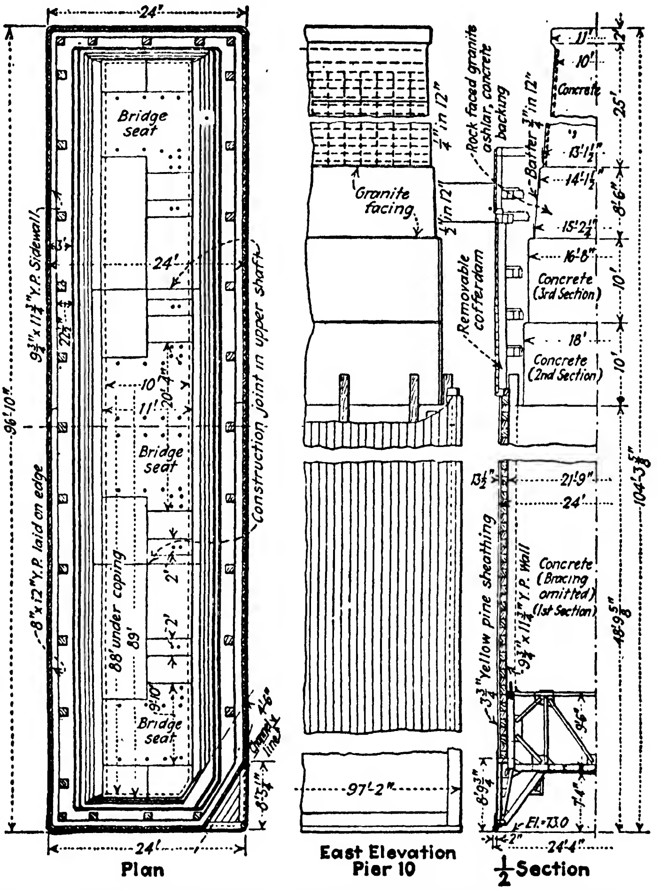


FIG. 10-6*d*.—Caisson of Newark Bay Bridge of the Central Railroad of New Jersey.

outside with 4-in. material. Above the crib there was a removable wooden cofferdam.

The caisson, together with the crib above, was built on its side at a shipbuilding yard. After the shaft was bulkheaded off, the caisson was launched, the steel working chamber riding lower in the water than the wooden superstructure. It was then towed to a drydock, righted by a derrick, and ballasted by depositing concrete in the wedge-shaped spaces in the walls of the working chamber. It was then floated to the pier site in its upright position and settled

to the bottom by adding concrete; after which it was sunk to rock in the usual manner.

In making studies for the caissons of the Delaware River bridge (1922), designs were made involving the use of timber, concrete, and a combination of steel and timber. The last was found to be the most economical. As a consequence, the sides and roof of the working chamber and the supporting trusses above the roof were made of steel, while the bulkheads, crib walls, and upper crib bracing were made of timber as shown in Fig. 10-6*e*.

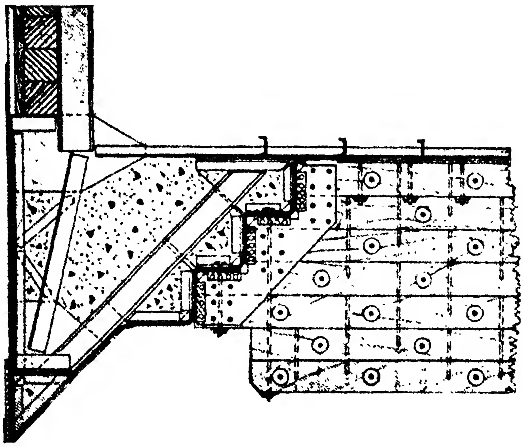


FIG. 10-6*e*.—Delaware River Bridge Caisson.

10-7. Pneumatic Caissons of Concrete. Relatively few examples are to be found in American practice where concrete caissons have been sunk their entire depth by the pneumatic process. However, many foundations have been placed by the open-caisson method (Art. 9-12) for a part of the depth, after which air locks were placed in the dredging wells and the final sinking done under air. In this way are combined the economy of open dredging with the reliability and certainty of results of the pneumatic process.

Figure 10-7*a* illustrates a vertical cross section—showing several stages of progress—of one of the record-depth caissons of the Kennebec River bridge at Bath, Maine, the largest of which was 25 by 54 ft. in plan. The deepest caisson had its cutting edge about $114\frac{1}{2}$ ft. below extreme high tide, and as excavation at one point was carried $10\frac{1}{2}$ ft. below the cutting edge, the total depth was 125 ft. The water depth at the site was about 40 ft., and the river bed consisted of coarse sand overlying a bed of plastic blue clay, under which was solid rock.

Each caisson was built to a height of 20 ft. on ways, all concrete being well reinforced, with the vertical rods fastened to the steel cutting edge. After providing for buoyancy, the caisson was launched, towed to the site of sinking, and moored in position; construction was then continued. The open-dredging method of

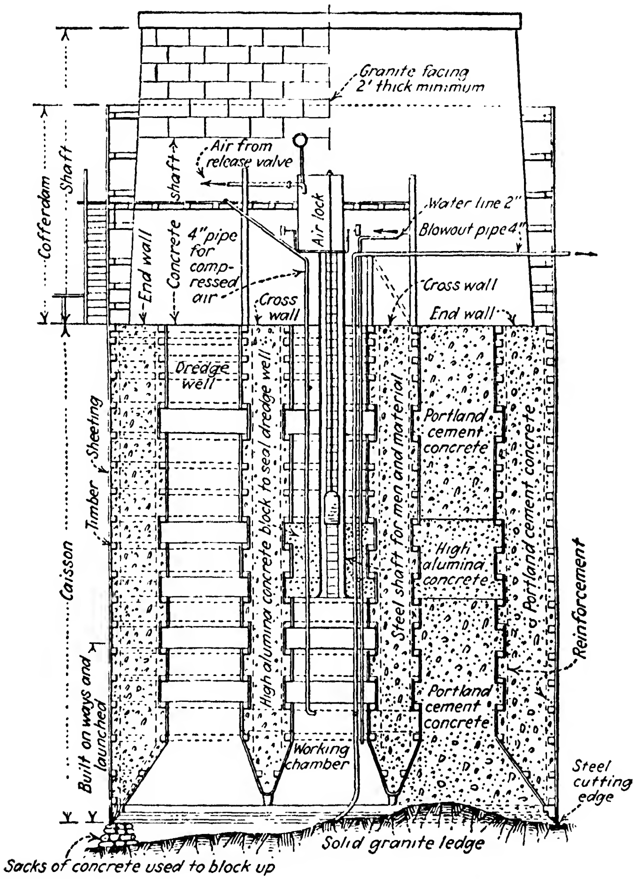


FIG. 10-7*a*.—Concrete Caisson of Kennebec River Bridge at Bath, Me.

sinking was used until the cutting edge penetrated the clay. Forms were then placed across the dredge wells and slabs of concrete, well reinforced and dovetailed into the sides of the wells, poured. After placing the shafts and air locks, the remainder of the sinking was done under air pressure.

10-8. Pneumatic Metal Cylinder Caissons. The foundation for a metal cylinder pier is often placed by the pneumatic process; in which case, like the open-cylinder caisson, there is usually no

particular point at which the caisson may be said to end and the pier begin. The pneumatic cylinder caisson is very similar to the open caisson in many cases, the only difference being that the former is fitted with horizontal diaphragm doors to form the air lock. Often a part of the sinking is done by the open-caisson method and the remainder by the pneumatic method. As noted in Art. 10-1, the cylinder caisson was the first type of foundation to which the pneumatic process of sinking was applied in this country.

Figure 10-8*a* illustrates the cylinder piers and pneumatic cylinder caissons used for the Columbia River bridge at Trail, British Columbia. The shells were of steel plates from $\frac{5}{16}$ to $\frac{7}{16}$ in. thick. The lower 61 ft. were formed of a double shell, the diameter of the inner shell being 3 ft. and that of the outer one 9 ft. at the bottom and 6 ft. at the top. Beginning at a point 8 ft. above the bottom of the caisson, the inner shell was splayed out to meet the outer shell at the cutting edge, thus forming a working chamber 8 ft. high. Near the bottom the two shells were braced together with diagonal lacing as shown in the diagram.

The upper parts of the cylinders were connected and braced by two vertical transverse $\frac{5}{16}$- by 60-in. plates, 2 ft. apart, braced together and the space between the two filled with concrete.

The air lock was formed by placing two diaphragm doors in the inner shaft, one about 13 ft. above the cutting edge and the other at a point 16 ft. higher. As sinking proceeded, a third door, about 16 ft. above the second door, was added, the upper two doors being used to form the lock.

With the cylinder caisson it is a simple matter to construct the cylinder to be used either as an open or as a pneumatic caisson. This makes it possible to utilize the advantages of both methods of sinking, the open caisson being used for that part of the sinking in which the material can be dredged or pumped out and the pneumatic process for that part where boulders or compact material are met and in finally preparing the foundation bed and placing the concrete filling in the working chamber.

The caissons for the Merrimac River bridge, between Salisbury and Newburyport, Mass., were of this type. Each caisson consisted of an 8-ft. diameter cast-iron shell, the metal being $1\frac{1}{2}$ in. thick and cast in 8-ft. sections. These sections had inside flanges bolted together, a mixture of red lead and linseed oil being placed between the joints.

The cylinders were sunk by inside dredging to a layer of boulders and gravel. They were then loaded with pig iron, air locks were

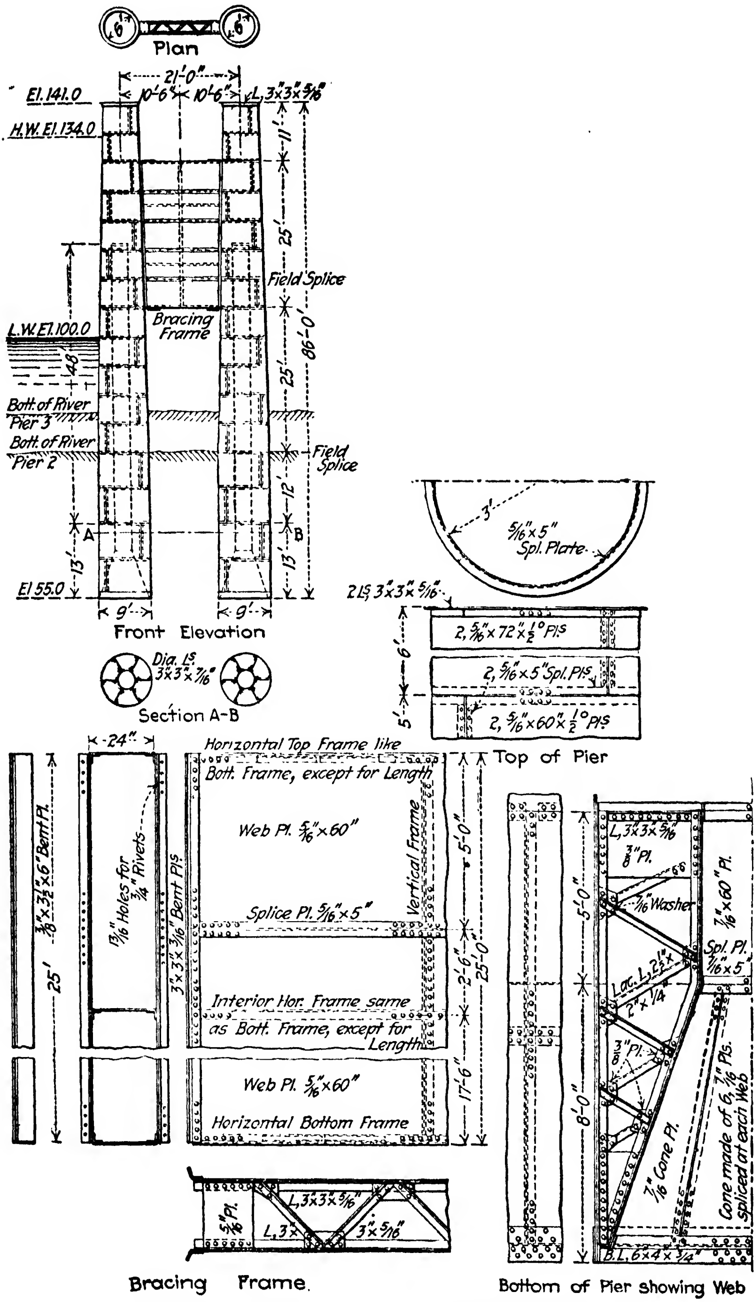


Fig. 10-8a.—Pneumatic Cylinder Caissons, Trail, B. C.

placed on top, and air pressure was applied. No attempt was made to sink the caissons through the boulders, but instead a novel method was used to transform this boulder and gravel layer into a good foundation bed. The pressure in the cylinder was reduced a little, allowing 1 ft. or more of water to rise. Portland cement was then mixed with the water to form a grout, which was kept well stirred while the air pressure was increased to force the grout into the gravel. When the grouting was completed, a depth of from 10 to 20 ft. of concrete was laid under air pressure and allowed to harden; the remainder was laid in the open.

10-9. Concrete Cylinder Caissons. Figure 10-9*a* illustrates the concrete pneumatic caissons used for the supports of a wharf and

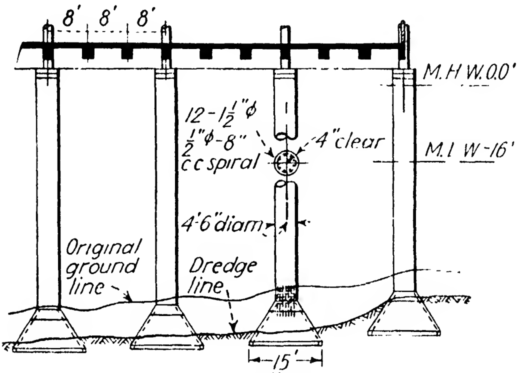


FIG. 10-9*a*.—Concrete Cylinder Caissons Supporting Wharf in Seattle, Wash.

shed of the American Can Company pier in Seattle, the maximum height of the caissons being about 70 ft. The shafts were $4\frac{1}{2}$ ft. in diameter, belled out in the bottom to a maximum diameter of 15 ft. The method of construction was to precast the hollow base with 6-in. walls, a 1:1:2 concrete mix being used. On top of this base section, semicircular sections of cylindrical steel forms were placed, the entire assembly then being picked up by a floating derrick and lowered into position. The reinforcing steel, consisting of twelve $1\frac{1}{2}$-in. round longitudinal rods and $\frac{1}{2}$-in. round spiral hooping at 8-in. centers, was made up in one continuous cage and lowered inside the steel shell. An 80-ton water-ballast tank surrounding an air lock was then bolted to the top of form, after which compressed air was applied to force the water out of the shell. Sandhogs entered, blocked the reinforcement in proper position, and mucked the cylinder down from 4 to 8 ft. through sand and gravel to firm

foundation, the bottom having been dredged of soft material prior to starting caisson work.

The 1:1:2 concrete filling was locked through at the top and allowed to fall the full height of the cylinder. Because of the uplift produced by the sloping sides of the base, the concrete on the base was allowed to set before the shaft concrete was placed. After the concrete had hardened, the steel shell was removed. Each cylinder was designed to carry a load of 800 tons, the permissible stress in the concrete being 675 lb. per sq. in. and the pressure on the foundation $4\frac{1}{2}$ tons per sq. ft.

The same general type of construction was used in building a pier at the Puget Sound Navy Yard at Bremerton, Wash., except that in

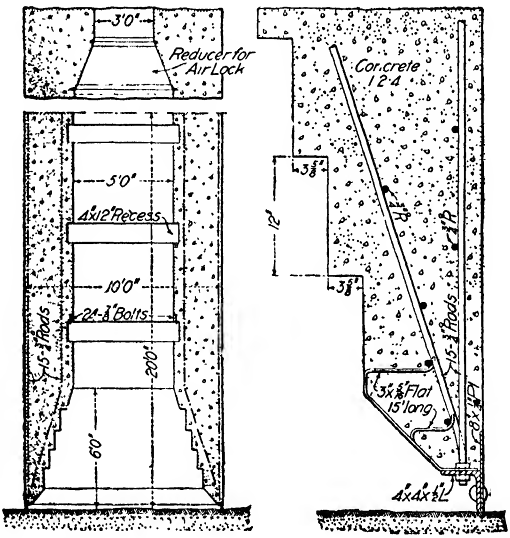


FIG. 10-9b.—Pneumatic Caissons of Reinforced Concrete for Bronx Viaduct of New York Connecting Railway.

place of the temporary steel shell, a permanent reinforced-concrete shell 6 in. thick was precast, quick-hardening cement being used to expedite the work. The casting and handling of these shells, which weighed up to 60 tons, proved to be a difficult task.

Figure 10-9b shows the main details of concrete cylinder caissons used for foundations of the Bronx viaduct of the New York Connecting Railway. The caissons varied from 10 to 18 ft. in diameter and were sunk to a maximum depth of 55 ft. The cutting edge was formed of a steel angle and steel plate, and the concrete composing the caisson was well reinforced with vertical and horizontal rods.

When sinking through clay, the open-dredging process was used, whereas in passing through quicksand, air locks were placed in the upper part of the shafts and the pneumatic process used.

10-10. Shafts and Air Locks. The shafts, which form the means of communication between the working chamber and the outside, are circular in shape and usually of steel plate about $\frac{3}{8}$ in. thick, with riveted or welded joints. The standard diameter is 36 in., but it may vary from 30 to 48 in. They are furnished in sections about 12 ft. long, each section having angle flanges at the ends, adjacent sections being bolted together through the outstanding legs of the angles, rubber gaskets being used to make the joint airtight. A

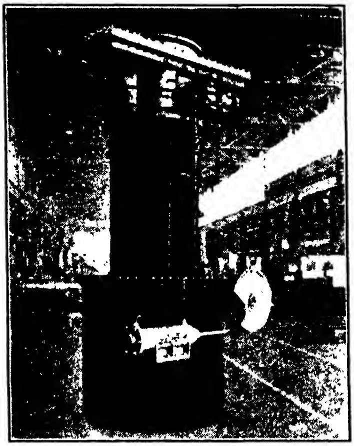


FIG. 10-10a.—Moran-Barr Type of Air Lock. (*Courtesy of Union Iron Works.*)

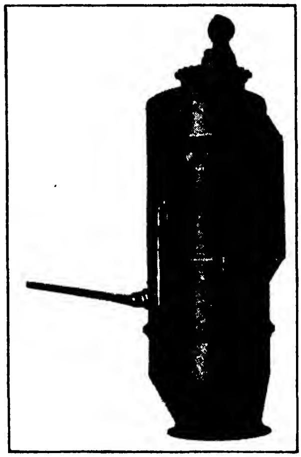


FIG. 10-10b.—Mattson Type of Air Lock. (*Courtesy of Union Iron Works.*)

ladder is built in the shaft if the same is to be used by the workmen. Where the depth is considerable, the men often use the excavating bucket as an elevator.

The air lock is a chamber with two doors, one opening to the atmosphere and the other to the shaft and working chamber. These doors are so placed that the unequal air pressure will always force them against their seats, which have rubber gaskets to prevent the escape of air. The operation of the lock for men is as follows: The lower door being closed and the upper one open, a man enters; the upper door is then closed and compressed air slowly admitted to the lock, and as soon as the pressure in it becomes equal to that below, the lower door opens, allowing the man to descend.

Figure 10-10*a* illustrates the Moran-Barr type of air lock as manufactured by the Union Iron Works. The top split doors slide

on adjustable rails and seat tight against a ground surface. The lower door is of the counterweighted, flap type. Both doors are operated by compressed air, although the lower door may be manually operated. The standard lock is built for 36- or 48-in. diameter shafting, but the top section and door openings are 36 in. net diameter. This lock is a development of the Moran air lock designed years ago by Daniel E. Moran and illustrated in Figs. 11-7*e* and *f*.

The Mattson type of air lock, as manufactured by the Union Iron Works, is illustrated in Fig. 10-10*b*. It differs from the Moran-Barr type by having the main or exit door on the side instead of at the top. The largest standard size is 50 by 96 in. The lower door opening is 36 in. in diameter and the side door is 38 in. wide and 48 in. high. In using this type, the bucket remains in the lock attached to the hoisting line and is discharged through the side door. This lock can be equipped with an air-motor hoist attached to the body of the lock, so that it is not necessary to use a crane or derrick.

FIG. 10-10*c*. Attachment for Placing Concrete. (*Courtesy of Union Iron Works.*)

Figure 10-10*c* shows an attachment that can be used with the Mattson air lock for the placing of concrete. The top cover plate carrying the sheave and rope stuffing box of the excavating lock is removed, and the concrete lock is inserted in the opening and bolted to the flange, which usually carries the stuffing box and sheave bearing plate. This concrete lock attachment is operated entirely outside of the material lock and does not require a man within the main lock to manipulate the bottom door of the attachment.

In the early caissons the locks were placed at the bottom of the shafts and extended down into the working chamber, but at present the locks are placed at the top of the shafts. Caisson sinking with the lock at the bottom is a risky undertaking because a "blowout," that is, a sudden outrush of air, will cause a like inrush of water accompanied by a rapid sinking of the caisson, which is almost sure to damage the lock. With the lock out of commission the men in the working chamber have no chance to escape; on the other hand, if the lock is at the top, the men can climb up and take refuge in the shaft above the level of the water. About the only disadvantage in having the lock on top of the shaft lies in the necessity of removing it each time a new section is added to the shaft; but with properly designed connections this can easily be done, and without danger, by

having an auxiliary door fitted to the lower end of the shaft in the roof of the working chamber which is closed when the lock is taken off.

10-11. Building and Placing the Caisson. The caisson may be built (*a*) on ways on shore, (*b*) on pontoons anchored near the shore or at the site, (*c*) on an artificial island, or (*d*) on a temporary platform, supported by piles, at the site. The first method is the one most often used where the physical conditions are satisfactory. These requirements are as follows: (*a*) there must be deep water

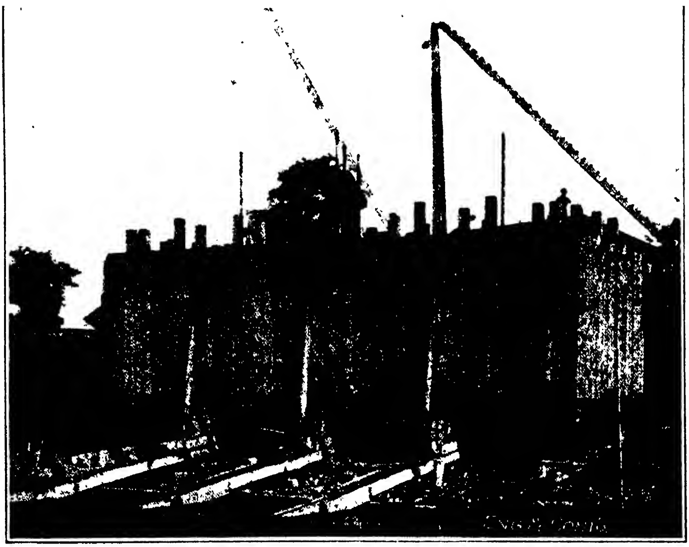


FIG. 10-11*a*.—Caisson on Launching Ways. Vancouver Bridge.

near the shore, (*b*) the ground must be firm enough to support the caisson, (*c*) there must be no danger of high water, and (*d*) the shore must not be too far from the site of sinking. Figure 10-11*a* shows the wooden caisson of one of the piers of the Vancouver bridge, Vancouver, Wash., as it was being built on launching ways.

A sufficient height of crib must be built previous to launching to ensure the top being well above water level. In the case of the caissons of the Vicksburg bridge, which were of steel for a height of 20 ft., on launching they drew 7 ft. and after placing concrete in the walls they drew 13 ft. Above the 20-ft. elevation the crib was of timber construction. Each foot of height of concrete placed in the crib increased the draft about $2\frac{1}{2}$ ft. Caissons are sometimes built with false bottoms to reduce the depth of immersion.

For large caissons in streams having considerable velocity, a pile dock or slip, open on the downstream end. is often constructed for holding and lining the structure while afloat after it has been towed to position. Special sea anchors are also often required in

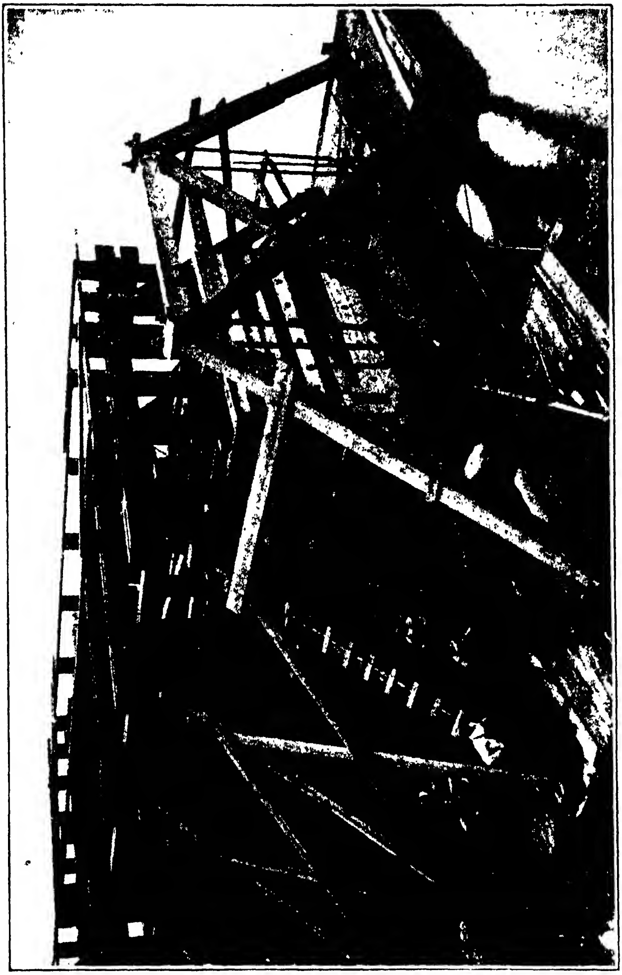


FIG. 10-11b.—Caisson Supported between Two Barges. Willamette River Bridge.

landing and holding caissons in place (see also Art. 9-14). One type used at the Vicksburg bridge consisted of a concrete mushroom about 7 ft. in diameter and $2\frac{1}{2}$ ft. high, built around a light-steel framework having a vertical member 7 ft. high with a shackle at the end for fastening the anchor cable. Another type of anchor consisted of a 16-in. H-beam from 18 to 30 ft. long, fitted with shackles

near the top for cable connections, driven into the river bottom nearly full length with a special type of follower. These were driven about 600 ft. away from the caisson.

Where satisfactory shore conditions are not available and the water is deep and subject to sudden rises, the pontoon method may be used. Sometimes two pontoons are used, and sometimes only one. Figure 10-11*b* shows one of the caissons of the Willamette River bridge of the Northern Pacific Railway as it was being built between two barges or pontoons. The caisson was held between

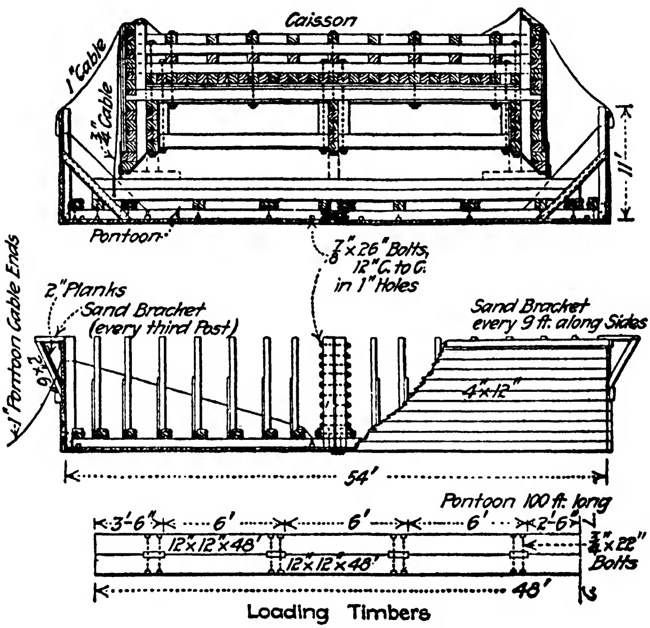


FIG. 10-11*c*.—Caisson-launching Pontoon, New Memphis Bridge.

the barges until a height of 20 ft. had been built up, when long screws were attached and the structure lowered into the water. Two heavy trusses, one at each end, tied the barges together to prevent any unequal motion from wave action.

The caissons for the new Memphis cantilever bridge were built in a single pontoon (Fig. 10-11*c*) and were launched by separating the 54- by 100-ft. pontoon into halves (after flooding it) and then pulling out the halves from under the caisson. The pontoon, which was constructed with 4- by 12-in. sheathing fastened to transverse 12- by 12-in. floor beams spaced 3-ft. centers, was built in halves connected along the longitudinal center line by a bolted joint. The contact surfaces of the halves were 12- by 12-in. edging sticks; and

$\frac{3}{4}$- and $\frac{7}{8}$-in. bolts held them together. Tar paper was placed in the joint to make it tight. The sides of the pontoon were 11 ft. high, and they were braced to the floor beams with 6- by 6-in. knee braces. Transverse deepened beams, made of two 12- by 12-in. timbers notched together, served as loading beams to distribute the caisson weight over the bottom of the pontoon through longitudinal timbers fastened to the transverse floor beams.

Pneumatic caissons are sometimes laid out and built at the site on artificial islands made by filling dredged material into areas surrounded by sheet piling. This method is particularly advantageous for large caissons of concrete, especially where the current has a high velocity. A modification of this is the sand-island method described in Art. 9-14.

A typical example of a small caisson built on a platform is as follows: Bearing piles are first driven in two longitudinal rows just clear of the caisson location. These are capped, and from the cap timbers are suspended transverse timbers by means of rods threaded their entire length and each provided with two nuts. Each transverse timber is held by means of a steel saddle on the underside, against which the lower nut of the rod bears and the other nut takes bearing on a washer on top of the pile cap. The transverse timbers are first screwed up against the underside of the cap timbers, and on these the caisson is built. After building up the crib, the caisson and transverse timbers are gradually lowered by unscrewing the nuts from the rods to permit the caisson to float in its correct position.

Figure 10-6*a* illustrates the tower bents supporting platforms, used in placing and guiding the steel caissons of the Caughnawaga Bridge described in Art. 10-6. The legs of the towers consisted of 12- by 12-in. timber spuds driven through square tubes made of $\frac{1}{8}$-in. bent plate, these tubes being preframed and braced together to form towers. Owing to the swift current, it was thought best to use land equipment in sinking the caissons, and to this end 1,500 ft. of trestle was built to reach the caisson sites. Caisson cutting edges, decks, and side walls were assembled at the shop in as large pieces as could be delivered by trucks and handled at the job. On arrival, working chambers were assembled on timbers spanning transversely across each caisson berth, after which the caissons were built to a height of 14 ft. and lowered into the water.

10-12. Sinking the Caisson. If mud covers the river bottom this should be dredged out before placing the caisson, as it is cheaper to remove it in this manner than to excavate it from within

the working chamber. Great care must be exercised in grounding the caisson to place it in its correct position. If in tidal water, this may be done by placing concrete in the crib to an amount which will just ground the caisson at low tide. Then the structure is placed in its true position at high tide and grounded as the water level lowers. Concrete is poured into the crib to an amount which will prevent floating when the tide rises. Often, where the caisson is slightly out of position, it may be floated by admitting a small amount of air into the working chamber. As soon as enough concrete has been placed to put on air pressure safely to expel the water from the working chamber, men enter to commence sinking operations. Sinking is effected by removing the spoil in the working chamber and by placing concrete in the crib. Water-jets are sometimes used to decrease side friction. When the caisson tends to "hang up," a temporary reduction of the air pressure in the working chamber may be helpful.

In clay the excavation may usually be kept some distance below the cutting edge, which offers the advantage of allowing more headroom for the men. This cannot be safely done in sand, as the water is very sensitive to changes of pressure, and so it is not possible to raise the pressure very much from that corresponding to the head on the cutting edge. In one of the caissons of the Rulo bridge a test well was sunk in clay 17 ft. below the cutting edge without any increase in the air pressure, but, when a 4-ft. vein of gravel was struck, the pressure had to be increased 8 to 10 lb. at once. In sinking a caisson for the Kinzie Street bridge in Chicago, the pressure was kept constant while sinking through a depth of 58 ft. in clay (see Fig. 10-12*a*).

In sinking caissons, the load is at first carried on the cutting edge, but, as the caisson gradually sinks, more of the load is resisted by friction on the sides and less by bearing on the cutting edge. Contrary to the usual custom, in the 55- by 180-ft. caisson of the New Quebec bridge (for details see Fig. 10-2*b*), which for the most part was sunk through sand, the caisson was supported on 40 sand jacks, about 25 posts of 12- by 12-in. southern pine, and 54 sets of blocking. The jacks and posts bore directly against the roof, while the blocking was piled under the bulkheads. When ready for a drop, the blocking and posts were first removed by washing the sand from under them with a water-jet; then the whole caisson was lowered by operating all the sand jacks simultaneously. Each jack consisted of a 29-in. steel cylinder closed at the bottom, having near the bottom two 3-in. holes with a sliding cover and a plunger

consisting of a single piece of timber fitting easily in the cylinder. The cylinder was first filled about two-thirds full of sand, the plunger inserted, and its upper end blocked against the roof. The operation of lowering consisted of opening the lower holes and washing out the sand with a jet.

The rate of sinking varies greatly, the larger the caisson and the denser the material, the slower will be the rate. Sinking operations

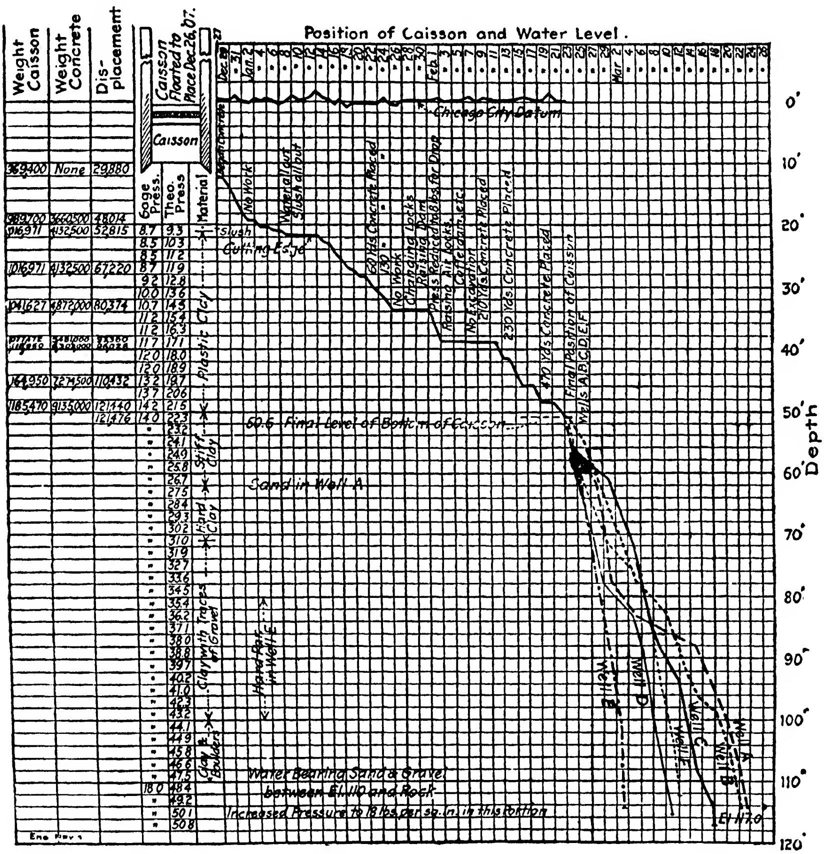


FIG. 10-12*a*.—Progress of Sinking Caissons, Kinzie Street Bridge, Chicago.

are usually carried on 24 hr. a day, and the rate will vary from zero to as much as 3 ft. a day. A chart is usually kept showing graphically the progress of sinking. Figure 10-12*a* shows the type of chart used in sinking one of the caissons of the Kinzie Street drawbridge, Chicago. The caisson is illustrated in Fig. 10-12*b*. Instead of carrying the whole caisson to bedrock, the cutting edge was stopped about halfway down, and wells were sunk the remainder of the

distance. However, this type of construction wherein the actual air pressure used varies greatly from the theoretical pressure is only possible in dense clay.

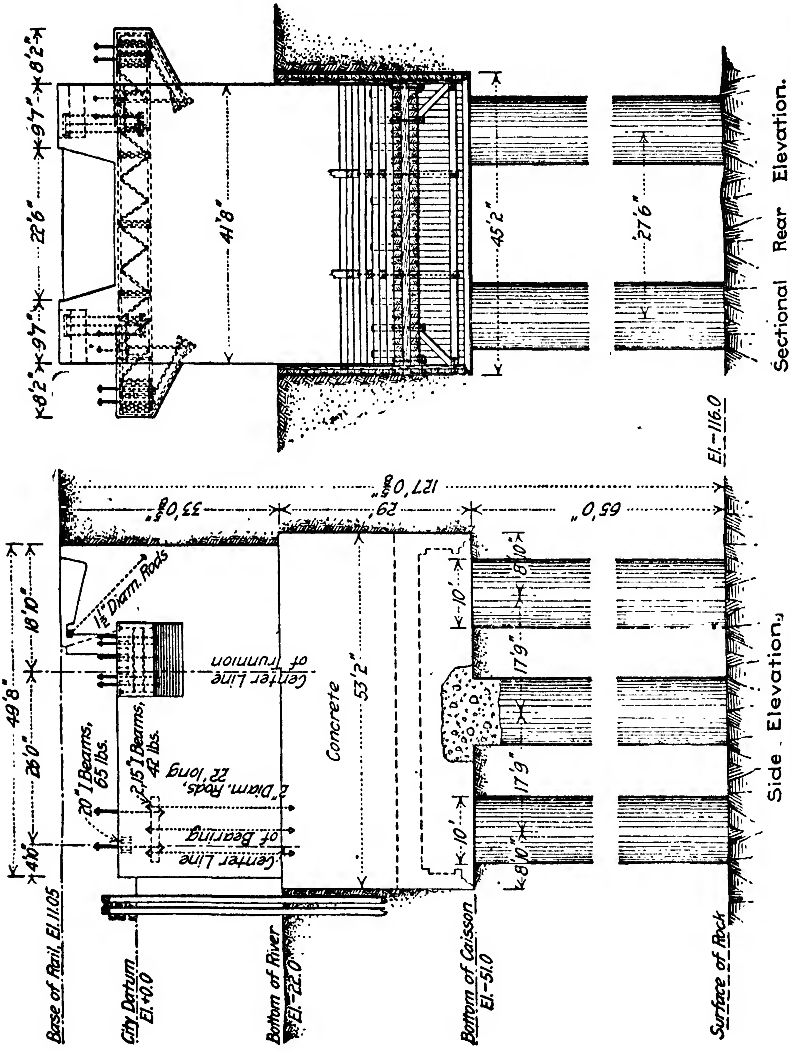


FIG. 10-12b.—Pneumatic Foundation for Chicago and North Western Railway Bridge, Kinzie Street, Chicago.

In sinking pier II of the Memphis bridge, excluding long delays, an average rate of 1.5 ft. per day of 24 hr. was maintained through sand and only 0.31 ft. through clay; on the other hand for piers 2 and 5 of the Thebes bridge, where hard gravel was encountered, the average rates were 0.23 and 0.41 ft., respectively. The rate per

day of sinking the St. Louis Municipal bridge caissons varied from an average of 0.68 ft. for pier 4 to 1.95 ft. for pier 3, with 1.28 ft. as an average for all caissons. The best progress in 1 day was 5.17 ft. For the caissons of the McKinley bridge, St. Louis, the average rate for all caissons was 2 ft. a day, with a maximum of 7.7 ft. in 1 day. For the Vicksburg bridge, one caisson was sunk 80 ft. through sand in 22 working days and another 75 ft. in 33 days. In sinking caissons for a Pennsylvania Railroad bridge over the Passaic River at Newark, N. J., the average rate was from 2 to $2\frac{1}{2}$ ft. a day.

10-13. Removing Spoil from Working Chamber. Various devices have been developed for removing the spoil from the air chamber. Where the material is sand, the blowout process or mud-and-sand pump is ordinarily employed; where clay is encountered, it is usually best to remove it with buckets, using some simple form of air lock, or perhaps the clay may be mixed with water and the sand-and-mud-pump process used. Boulders must be removed through the air locks.

The blowout process is a very simple affair, the principle consisting of using the pressure in the air chamber to drive out sand or mud when it is piled around the inlet of a pipe which leads from the working chamber to the open air. The diameter of the pipe is usually about 4 or 5 in., the top being fitted with an elbow to throw the sand in a horizontal direction, while to the lower part is attached a flexible hose of large diameter with a valve. To blow out the sand and mud, it is only necessary to heap it up around the mouth and to open the valve; the material is then carried out with a high velocity, in fact, the velocity is so great that the pipe rapidly wears away. At the Havre de Grace bridge the elbow, which was of chilled iron, 4 in. thick, was worn through in 2 days. Many contractors use very hard manganese steel for these elbows. Considerable care must be exercised in placing the material against the inlet, for, if a considerable amount of air is not admitted with the sand and mud, it will clog, or, if there is too much air admitted, it is a waste. It has been found advantageous to have small holes in the pipe above the inlet, as this gives more uniform action, tending to draw the material up instead of merely driving it and thus lessening the amount of air entering with the sand and mud.

In the construction of the Waverly bridge over the Missouri River a T-section was used instead of a gooseneck, the vertical pipe extending up beyond the point of discharge. When the top of this pipe was capped, the material that was discharged

encountered an air cushion which eliminated the heavy wear at the bend.

The dry blowout process is a very rapid and satisfactory means of removing spoil from the working chamber, although the consequent lowering of pressure in the air chamber causes a thick fog and also may risk entrance of water from the outside. If too much air is pumped into the working chamber, a blowout results, which is followed by a sudden inrush of water under the cutting edge. Care must be exercised to keep the pressure reasonably constant. The dry blowout process is most satisfactory under fairly low pressures, although a head of at least 20 ft. is necessary. This process was first used by William Sooy Smith in 1869 in building bridge piers in the Savannah River. When a stratum was reached that was so nearly impermeable that the air pressure in the working chamber would not force out the water, a pipe was run down from above. On reaching a layer of sand it was found that the sand as well as the water was rapidly driven through the pipe.

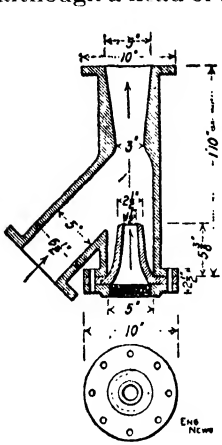


Fig. 10-13*a*.—Details of Hydraulic Ejector.

The principle of the sand-and-mud pump, or hydraulic ejector, is that of the induced current, where a quantity of water with a high velocity causes a reduction of pressure which draws the mud and sand—well mixed with water—into the pipe. Figure 10-13*a* illustrates the form often used. The water enters through a pipe at the bottom under a high pressure and passes up through the small nozzle, at the exit of which, on account of the high velocity, the pressure is low. The opening on the left connects with a pipe or hose, the lower end of which rests in a pool of mud or sand and water. On account of the difference of pressure at the two ends of this pipe the mud is drawn into the pump and carried upward with the water through a pipe which connects with the top of the pump. The essential difference between this form of excavator and the blowout process is that in the former the water is the moving force doing the work, while in the latter it is the air from the working chamber. The water pressure used is ordinarily about 80 lb. per sq. in. This method was first used by James B. Eads in the caissons of the St. Louis arch bridge.

In the Williamsburg bridge, New York, the hose was extended to a sort of sump in the bottom of the excavation where its open end was placed below the surface of the water. Gravel, sand, and mud were constantly fed into the nozzle by a laborer who raked it up and prevented the hose from clogging, and another man with a $\frac{3}{4}$-in. nozzle played a 50-lb. water-jet against the soil to wash it into the sump.

In some caisson work at Arran, Switzerland, instead of using a sump, a horizontal hopper was employed, the discharge pipe leading from the lowest point in the hopper. A jet of water from a small pipe was constantly played on the material as it was fed into the hopper.

Clay is usually more cheaply removed with buckets than by any other method. Large rocks must be blasted to pieces and removed with buckets.

A novel device, called the "water column," was used in the caissons of the Brooklyn bridge to remove the material. It consisted of an open shaft, the lower part extending into a sump which was kept full of water and the shaft itself was filled with water up to a point sufficient to balance the air pressure in the caisson. Workmen pushed the spoil under the shaft, and from there it was removed by dredging with an orange-peel or clamshell bucket.

The same device was used in 1921 in caisson work for a bridge across the Fuerte River on the Southern Pacific Railroad in Mexico. Here the shaft was $5\frac{1}{2}$ ft. in diameter, extending $2\frac{1}{2}$ ft. below the cutting edge of the caisson.

10-14. Concreting the Air Chamber. When rock is reached, if the same is level, it is only necessary to clean off all loose material before depositing the concrete. On the other hand, if the rock is not level, some preliminary work must be done; if the rock has a uniform slope, it should either be blasted down to a level surface or else stepped, unless very rough; although, if the rock surface is at practically the same elevation all around the cutting edge of the caisson, but irregular within, little more than a thorough cleaning will be necessary. For those caissons founded on clay or hardpan, a level surface is easily obtained.

Caisson 10 of the Passyunk Avenue bridge landed on rock which had a slope of about 5 ft. in the length of the caisson. As soon as rock on the high side was reached, the cutting edge on the low side was blocked with 6- by 12-in. timbers, 6 ft. apart, after which excavation under the cutting edge was carried to rock and extended

$2\frac{1}{2}$ ft. out beyond the cutting edge. This excavation was then filled with concrete.

The rock under the caissons of the Kennebec River bridge (Art. 10-7) was so irregular that in some cases the extreme difference of elevation of rock was as much as 13 ft. On the cutting edge reaching the high point of the rock, a small area under the cutting edge was excavated and cleaned off to the surface of the ledge and if necessary leveled off. Sacks of concrete (see Fig. 10-7*a*), mixed of quick-hardening cement, were then laid up as a pier to give bearing under the cutting edge. After completion of the blocking, the entire surface of the base was cleaned, washed, and scrubbed with brooms. Prior to placing the concrete filling, several cubic yards of grout was placed over the bottom.

In the caissons for the St. Louis Municipal bridge the rock surface was irregular, but no attempt was made to level it off or to bring the caissons to bearing throughout. Where depressions occurred, the sand was removed, and sacks of concrete were deposited on the rock and tamped under the cutting edge, after which concrete was placed in the working chamber in the usual manner.

In placing foundations for the $18\frac{1}{2}$- by 87-ft. pier of the new Thames River bridge at New London, Conn., because of the considerable slope of the rock in the upstream direction—approximately 35 ft. in 90 ft.—three 22-ft. diameter caissons were used in place of one large one, the spacing being 35 ft. center to center.

The concrete for filling the working chamber may be carried in through the material shafts and locks by means of buckets, or special arrangements may be made, by placing a concrete-lock attachment on the air lock (see Art. 10-10). In using this, concrete is placed through the open door in the top of the attachment. This door is then closed and air admitted to the lock, allowing the lower door to open and the mass of concrete to fall through the shaft to the working chamber.

In filling the working chamber with concrete great care must be exercised to fill the chamber completely. To accomplish this, concrete is placed to within about a foot of the roof, the air pressure being gradually reduced according to the change in head from the cutting edge upward. Work is then suspended for at least 24 hr. under air pressure to allow the concrete to set and to attain its maximum shrinkage. A wet concrete should be used, as the moisture is rapidly absorbed in compressed air. The air lock is then removed and very wet concrete—or grout in same cases—is dumped down the shaft to fill the remainder of the chamber and the lower

part of the shaft. Air vents 1 or 2 in. in diameter are commonly built in the caisson as far as possible from the shaft used in placing the concrete. If the concreting has been properly done, these vents will be found to be filled with grout on completion of the work.

10-15. Frictional Resistance. Estimating the probable frictional resistance to be met with in sinking caissons is one of the most difficult features involved in the design. It depends upon numerous factors, such as the kind of material penetrated, the material com-

TABLE 10-15*a*. SKIN FRICTION FOR PNEUMATIC CAISSONS OF BRIDGES*
(Expressed in Pounds per Square Foot)

Name of bridge	Range for separate piers	Average	No. of piers	Materials penetrated in sinking caissons
Bellefontaine. . .	600–700	648	4	Fine sand, sand, coarse sand, boulders
Blair Crossing. . .	330–410	381	4	Fine sand, coarse sand, clay
Brooklyn.	600			
Cairo.	622–932	750	10	Sand
Havre de Grace.	308–489	400	4	Silt, sand, mud
McKinley.	600			
Memphis.	365–837	584	5	Sand, gravel, mud, clay, sediment, very tough clay, quicksand
Miles Glacier. . . .	620			
Nebraska City. .	409–590	525	3	
New Omaha. . . .	472–673	617	5	Sand, gravel, some clay to bedrock
Rulo.	351–944	614	4	River sand, coarse sand, rubbish, clay, gravel
Sioux City.	314–535	463	4	Fine sand, yellow sand, gravel, clay, boulders
Williamsburg. . . .	750			

General average for nine bridges, 554 lb. per sq. ft.
* See *Trans. A.S.C.E.*, vol. 62, p. 113, March, 1909.

posing the sides of caisson and crib, the depth to which sunk, whether the sides of the caisson are vertical or flared, whether or not the water-jet is used, and the amount of air leaking under the cutting edge.

In general the frictional resistance per square foot of exposed surface of caisson and crib will seldom be less than 250 or more than 1,000 lb. Next to mud and silt, sandy soils offer the least resistance, especially when carrying large amounts of water, but clay will offer less resistance than material containing boulders. With uniform

soil conditions, the unit friction will increase with the depth; for example, at the McKinley bridge, which crosses the Mississippi River at St. Louis, the friction was found to be about 300 lb. per sq. ft. of exposed surface at 40 ft. and 600 lb. at a penetration of 70 ft.

Anything which tends to loosen the soil around the sides of the caisson and crib will decrease the friction, at least for a short time; escaping air has about the same effect as the water-jet in lubricating the material. Although flaring out the bottom of the caisson tends to reduce the side friction, yet, on account of possible wedging

TABLE 10-15*b**

No.	Type of caisson	Method of sinking	Material penetrated	Skin friction	Depth below low water in feet	Area of base in square feet
1	Cast iron	Open excavation	Gravel, clay	240	60	125
2	Cast iron	Open excavation	Sand, clay	250	75	225
3	Cast iron	Open excavation	Sand	250	60	125
4	Wrought iron	Open excavation	Sand, clay	285	140	1,000
5	Cast iron	Open excavation	Sand, clay, gravel	300	100	125
6	Cast iron	Open excavation	Sand	325	60	125
7	Cast iron	Open excavation	Silt	350	60	125
8	Steel construction	Open excavation	Silt, sand, clay	375	55	190
9	Cast iron	Open excavation	Silt, mud, clay	390	75	100
10	Timber construction	Open excavation	Sand	450	30	1,300
11	Steel construction	Open excavation	Silt, clay	450	60	700
12	Steel construction	Open excavation	Silt, clay, sand	450	60	1,200
13	Steel construction	Open excavation	Mud, sand	450	65	1,300
14	Steel construction	Open excavation	Clay	450	75	1,500
15	Iron construction	Open excavation	Sand, gravel, clay	480	65	200
16	Cast iron	Open excavation	Clay	500	60	125
17	Steel construction	Open excavation	Clay	700	65	1,300
18	Masonry	Pneumatic	Sand, mud	205	40	75
19	Timber construction	Pneumatic	Clay	250	35	800
20	Steel construction	Pneumatic	Clay, sand	275	60	150
21	Timber construction	Pneumatic	Silt, sand, mud	310	75	2,550
22	Timber construction	Pneumatic	Sand, clay, gravel	350	100	1,200
23	Timber construction	Pneumatic	Sand, clay, boulders	400	48	1,925
24	Timber construction	Pneumatic	Clay, sand, gravel	400	95	4,500
25	Timber construction	Pneumatic	Sand, gravel, clay	425	55	1,300
26	Steel construction	Pneumatic	Sand, boulders	450	68	2,700
27	Timber	Pneumatic	Silt, clay, gravel	500	75	1,800
28	Iron cylinder	Pneumatic	Sand, shale	525	60	1,200
29	Timber construction	Pneumatic	Sand	540	75	1,700
30	Timber construction	Pneumatic	Sand, clay	600	75	1,400
31	Timber construction	Pneumatic	Sand, gravel, clay	650	80	2,000
32	Timber construction	Pneumatic	Sand	650	90	1,200
33	Timber construction	Pneumatic	Sand, boulders	660	101	2,100
34	Timber construction	Pneumatic	Silt, sand, clay	900	45	1,700

* *Eng. Contracting*, vol. 56, p. 172, Aug. 24, 1921.

action by material falling into the open space above the bottom and, further, on account of the loss of guidance, pneumatic caissons are now practically all made with vertical outside walls. Care should be taken to prevent the caisson from warping, for, if the four sides are not in true planes, the friction on the sides is greatly increased.

Table 10-15*a* gives values for the skin friction on pneumatic caissons when they were well down for a number of notable structures, while Table 10-15*b* gives frictional values for both open and pneumatic caissons; values are in pounds per square foot of caisson surface in contact with the earth.

In sinking the caissons for the Commercial Cable Building in New York City, the frictional resistance varied from 250 to 300 lb. per sq. ft. of exposed surface, but in the United Fire Insurance Company caissons in the same city it was as high as 1,000 lb. per sq. ft. In testing the frictional resistance of a 4-ft. diameter pier sunk 65 ft. below ground level in Chicago by the Chicago method (Art. 12-2), the material under the pier was completely removed, and on placing load on the pier, the first evidence of movement was at 300 lb. per sq. ft. and at 700 lb. per sq. ft. the movement continued indefinitely. In sinking 12-ft. square caissons through fairly soft blue clay in Detroit, the frictional resistance was 450 lb. per sq. ft. One of the highest values ever observed was 1,912 lb. per sq. ft.

C. E. Fowler gives a table of frictional values expressed in pounds per square foot of exposed surface for several soils and for depths varying from 25 to 125 ft. For these two limiting depths they are as follows: silt or soft mud, 150 to 450; stiff mud, 300 to 600; packed sand, 750 to 1,350; clay or ordinary gravel, 900 to 1,500; heavy packed gravel, 1,050 to 1,650; and hardpan or cemented gravel, 1,350 to 1,950.

10-16. Physiological Effects of Compressed Air. In the early years almost all important work employing compressed-air workers levied a heavy toll of suffering and death on the "sand hogs," as caisson workers are commonly called. For example, on the caisson work of the St. Louis arch bridge (1869) there were 119 cases of caisson disease and 14 deaths. In contrast to this, on the Vicksburg bridge job (1929) there was not a single death or serious injury, in spite of the fact that the air pressure reached the record figure of 53 lb. per sq. in.

Usually no harmful effects are felt on entering compressed air or while remaining in it, although occasionally eardrums are broken and blood vessels ruptured. Experienced men often find it impos-

sible to enter when troubled with a bad cold. Trouble usually comes during decompression. The attack may be light, or it may be severe. A light attack is characterized by severe pains, chiefly in the joints, and closely resembles rheumatism in its effects. From its tendency to cause its victims to double up in agony, it is commonly known as the "bends." When the attack is very severe, it usually paralyzes its victim and is often fatal.

Some of the sensations felt on entering compressed air are heat, slight giddiness, inability to whisper, and a feeling of resistance to movement owing to the density of the air. Pain may be felt in the ears, which may be relieved by closing the mouth and holding the nose, and at the same time trying to expel the air from the lungs; such action makes the pressure in the tympanic cavity equal to the outside pressure by means of the Eustachian tubes, which run from the back of the nasal passages to the cavity. Relief may also be secured by swallowing.

On leaving the air pressure, one feels cold, this sensation being keenest during the passage through the air lock. It is due to the expansion of the air in the lock, as well as to the expansion and liberation of gases in the body. To counteract the effects of this cold, the air lock should be heated, the men should be given strong hot coffee to drink on emerging, and they should dress warmly. Another sensation often manifested on emerging is an itching, pricking feeling under the skin on all parts of the body; this disappears in a few minutes.

If a person is taken with caisson illness, a variety of symptoms may be present. First among these are neuralgic pains of varying severity. Another characteristic symptom which is always exhibited is a profuse cold perspiration. There may be pain at the pit of the stomach, usually, but not always, accompanied by vomiting. Paralysis may appear, the degree varying from slightly impaired sensation or numbness in the extremities to a total loss of sensation and motion in the affected parts, which are most frequently the legs and lower part of the body. Usually there are dizziness, double vision, incoherence of speech, and sometimes unconsciousness. The duration of these symptoms is from a few hours to a number of weeks. In fatal cases congestion of the brain and spinal cord always exists.

10-17. Cause of Caisson Disease. It is said that attention was first called to caisson disease at about the middle of the last century by Triger, who applied the use of compressed air in sinking some shafts at Chalons on the banks of the Loire. Hoppe Seyler (1857)

and Thomas Schwann (1858), in Germany, and Busquoy (1861), in France, gave the first correct suggestion as to its cause. But it was Paul Bert, who, by his remarkable experiments published in 1878, proved the true cause of caisson disease to be the effervescence of gas in the blood and tissue juices.

The gas which is present in the blood, and which comes out of solution if the pressure is too rapidly lowered, is mostly nitrogen; for, if the working chamber is properly ventilated, there will be only a small amount of carbonic acid gas in the air, while the oxygen content dissolved by the blood is taken up chemically by the hemoglobin. The tissue fluids, chiefly the blood, dissolve the air according to the well-known physical law that the amount of gas dissolved in a fluid is proportional to the pressure of the gas surrounding the fluid.

For pressures used in pneumatic-caisson work, which seldom exceed 50 lb. per sq. in. gage pressure, these dissolved gases probably have no chemical effect on the system and are quite harmless as long as they remain in solution. In diving work, depths of 250 ft. or more, corresponding to gage pressures of 108.5 lb. and over, have been reached. As stated in Art. 9-13, divers inspected caisson work on the San Francisco-Oakland bridge at a depth of 220 ft.

If the air pressure is lowered slowly, the gases are thrown out of the blood at the lungs without developing bubbles of any appreciable size. But if the pressure is lowered too rapidly, the gas bubbles, owing to their size, stick in the minute blood vessels and obstruct the flow of the blood, often causing the vessels to burst. The same condition obtains in the various tissues carrying juices saturated with gas. If these bubbles develop in the joints, the result is the bends; if in the spinal cord, paralysis; and if in the heart, heart failure.

10-18. Prevention of, and Cure for, Caisson Disease. If the cause of caisson disease is a mechanical action due to the development of bubbles in the blood and fluid tissues, which in turn is due to too rapid decompression, then manifestly the cure is decompression at a rate slow enough to avoid this phenomenon. The length of time will depend upon the amount of gas in the fluid tissues and upon the physical characteristics of the person being decompressed. The amount of gas in the fluid tissues will, in turn, depend upon (*a*) the degree of pressure in the working chamber and (*b*) the length of time under pressure. The length of time taken to saturate the body fluids at any particular pressure will vary greatly, depending upon the fatness of the subject, the amount of bodily work done, heat and

moisture present, etc. From experiments, Haldane concluded that, in certain parts of the body where the circulation is rapid and the number of blood vessels high, the tissue juices will become 50 per cent saturated in 5 min., with complete saturation in 40 min.; whereas other parts, lacking a copious supply of blood, will require 75 min. for 50 per cent saturation and about 4 hr. for 90 per cent saturation. Experiments show that the fatty tissues absorb about five times as much gas as does the blood and the rate of absorption is much slower; the rate of desaturation will be correspondingly slow. For this reason most authorities state that men inclined toward fatness should never be employed for compressed-air work. The better the circulation of the blood, the more quickly and easily will the gases be thrown out of the system; for this reason only men in good physical condition should be employed. Old men, or those who have abused themselves by excessive drinking or other dissipation, should never be allowed in the working chamber.

Edward Levy, after a study of nearly 2,000,000 decompressions in tunnel work accompanied by 4,372 cases of caisson disease, thinks that normal lungs, normal kidneys, and a good heart are the important things and that age and moderate fatness are probably not so detrimental as some believe.[1]

Authorities differ as to the time that should be allowed for decompression. Some urge a uniform rate of decompression, while others prefer stage decompression, that is, at first a rapid decompression to a certain pressure, followed by slower decompression.

Seldom is sufficient time given to decompression, because the air lock is small and as a consequence the men must maintain cramped positions in the same. Moreover, the lock is usually cold and filled with fog, due to the decreasing pressure. Properly, the lock should be large enough to allow the men some freedom of motion, and it should be ventilated with warm, dry air. The French law prescribes that for a head of water up to 65.6 ft. not less than 21.2 cu. ft. of air shall be provided for each man in the lock and for depths above this not less than 24.7 cu. ft. During decompression the men should constantly move about and massage their various joints, as this has been found to assist materially in ridding the system of the gases.

MacLeod suggests the time for decompression as being safe is as shown in the table on page 357.

In Germany, von Schrötter, Heller, and Mager published a work in which they laid down the principle that a uniform decompression

[1] *U. S. Bur. Mines, Tech. Paper* 285.

at the rate of 2 min. per 0.1 atmosphere, or 20 min. per atmosphere, was safe.

Gage pressure	Length of shift, hours	Decompression period, hours
15–30	4	$\frac{1}{2}$–1
45–60	4	$1\frac{1}{2}$–2

The theory upon which stage decompression is based is that the gas in the blood will not effervesce until a marked diminution of pressure obtains, and, since, to the point of effervescence, the gases are discharged at a rate varying with some function of the change of pressure, manifestly the more rapid the lowering of pressure, the more quickly will the blood vessels be freed of the gases contained therein. Since almost no cases of caisson disease are caused by rapid decompression from about 19 lb. gage pressure, it seems reasonable to assume that the pressure in the air lock may be reduced 19 lb. per sq. in. in about 3 min.; from this point the pressure must be lowered quite slowly and should correspond to the natural rate of desaturation of the fluid tissues at that difference of pressure.

In his study previously noted, Levy found that with men working under the New York regulations there was no caisson disease with pressures under 15 lb. Up to a pressure of 29 lb. the number of cases increased at a fairly uniform rate, 248 cases per 10,000 decompressions being the rate at 29 lb. where there were two 3-hr. shifts per day. At 30 lb. pressure and two 2-hr. periods the rate decreased to 49 per 10,000 decompressions. The rate at 34 lb. pressure and two 2-hr. shifts was 150 per 10,000 as compared with a rate of 77 where the pressure was 35 lb. and the daily working time two $1\frac{1}{2}$-hr. periods. This shows clearly the effect of the length of working periods.

Apart from the matter of slow decompression, other precautions, if taken, will do much to lessen the occurrence of caisson disease. Anything which tends to lower the vital resistance of the human system tends to promote caisson illness. For this reason the physical conditions under which the men work should be as good as it is possible to make them: There should be furnished plenty of fresh air;

electric lighting rather than gas or candle lighting should always be employed, as the latter tends to vitiate the air; the air should be kept at as reasonable a temperature as possible, which means that it should be cooled during the summertime, as compression raises its temperature. At present this is done in practically all work, either by spraying the compressed air as it enters the working chamber or else by passing it through a coil of pipes externally cooled.

The air should be kept pure, especially when sinking through foul material. T. K. Thomson reports that, when sinking through the foul bottom of the Harlem River, the men suffered much from the bends, but, when sinking through the clay below this, even though under a much greater pressure, very little trouble occurred. It is also noticed that a greater amount of sickness is apt to occur during concreting than at other times, this being due to the decrease in the leakage of the air, or inadequate ventilation.

In recent years tests have been made on a mixture of helium and oxygen to be used by divers instead of air. Helium has a lower coefficient of solubility and a greater diffusivity than nitrogen; consequently its use should make possible a shorter required time for decompression. Experiments indicate that this time may be reduced to one-third or one-fourth that required when air is used. Experiments have also been carried out on caisson workers in the use of pure oxygen during decompression. These experiments indicate that in this manner nitrogen taken into the body during work is relieved more rapidly without the formation of nitrogen bubbles.

The best and about the only cure for caisson disease is recompression with slow decompression. If the patient can be put into the air before the gas bubbles have had a chance to tear the blood vessels and fluid tissues, a cure can usually be effected, but otherwise not. For this reason, a hospital air lock, large and well ventilated, should always be maintained in readiness, and the men should be housed near by, so that in case of delayed attacks they may be immediately recompressed.

This method of treatment probably dates from the time when E. W. Moir, in 1890, introduced the medical lock for decompression in connection with the construction of the first tunnels under the North River, New York.

10-19. Rules for Compressed-air Workers. The rules of New York State (1938) governing the time of decompression are considered to be quite satisfactory in protecting the workers. It is specified that decompression must give a drop of half the maximum gage pressure at the rate of 5 lb. per min., the remaining decom-

pression being at a uniform rate of such value that the total time will not be less than that required by the following table:[1]

Gage pressure, pounds per square inch	0–15	15–20	20–30	30 and up
Average rate, pounds per minute (not more)	3	2	1.5	1

For example, if the maximum gage pressure is 40 lb., the first 20 lb. may be dropped at the rate of 5 lb. per min., requiring 4 min.; hence the remaining 20 lb. drop will require 36 min., since the total required time is 40 min. Pressures in excess of 50 lb. may not be used except in case of an emergency.

The New York rules provide that the working time in any 24 hr. shall be divided into two shifts under compressed air and an interval in the open air, as follows:

Gage pressure, pounds	0–18	18–26	26–33	33–38	38–43	43–48	48–50
Time per shift, hours, maximum	4	3	2	$1\frac{1}{2}$	1	$\frac{3}{4}$	$\frac{1}{2}$
Total time, hours, maximum	8	6	4	3	2	$1\frac{1}{2}$	1
Rest interval, hours, minimum	$\frac{1}{2}$	1	2	3	4	5	6

New Jersey and Pennsylvania allow practically the same working hours as New York, but the time required for decompression is somewhat less—probably too low—particularly in the case of Pennsylvania, where the rates are as follows:

Gage pressure, pounds	10–15	15–20	20–25	25–30	30–36	36–40	40–50
Time of decompression, minutes	2	5	10	12	15	20	25

In sinking the pier caissons of the Delaware River bridge in 1922, there were 16.2 cases of bends per 10,000 decompressions for the Philadelphia side and 31.2 for the Camden side. The maximum depth of sinking below mean high water for the Philadelphia caisson was 58 ft. and for the Camden caisson 82 ft. The time required for locking out from 15, 20, 30, and 35 lb. was 5, 10, 18, and 25 min., respectively. The working period at pressures less than 20 lb. was 8 hr. in two shifts of 4 hr. each, with a 1-hr. intermission between shifts; from 20 to 30 lb., 6 hr. in two shifts, with an intermission of 3 hr.; and from 30 to 35 lb., two shifts of 2 hr. each, with a 2-hr. rest period between shifts. On this work there were no fatal cases.

[1] State of New York, Dept. Labor, *Industrial Code Bull.* 22.

In the caisson work for the Grey Street bridge in Brisbane, Australia, in 1932 where pressures of 50 lb. were used, the men were locked in at the rate of 2 lb. per min. They were locked out in accordance with the following rule: decompression to half the gage pressure at 2 lb. per min., then 5 lb. at the rate of 1 lb. per min., then 5 lb. at the rate of $\frac{1}{2}$ lb. per min., then to atmospheric pressure at the rate of $\frac{1}{3}$ lb. per min.

So little work has been done by divers at depths of 250 ft. that a good deal of uncertainty exists as to safe rates of decompression for divers. For depths of over 200 ft. the diver may come up to a depth of 100 ft. and stay there 3 min. Stops of the same length may be made at depths of 75, 60, 45, and 30 ft., after which he comes to the surface and enters a recompression chamber in which the pressure is 50 lb. He remains in this chamber for 2 hr. during which the pressure is gradually reduced. For a depth of 250 ft. the maximum time a diver may stay on the bottom is about 15 min.

CHAPTER XI

PNEUMATIC CAISSONS FOR BUILDINGS

11-1. General Development. The application of the pneumatic caisson to building foundations has been restricted very largely to the tall buildings or "skyscrapers" of a few large cities of which New York City is a notable example. Two conditions occur there which require this form of foundation: (*a*) the necessity for carrying the column loads to bedrock and (*b*) the presence of quicksand over the rock. Both the height of the buildings and the magnitude of the column loads make it imperative to found the piers on a very hard and unyielding stratum, preferably bedrock, since any irregular settlement is exceedingly dangerous and difficult to remedy in tall buildings. The presence of quicksand makes sinking to bedrock very difficult by other methods than that of the pneumatic caisson, due to the tendency of the material to flow into the excavation; it is especially dangerous in the lower part of Manhattan Island, due to the liability of undermining adjacent building foundations, many of which rest on shallow foundations. The disadvantage of the pneumatic method is its high cost, but this is justified where the security of very expensive buildings is at stake. In 1927 the first deep open foundation installation was made in lower New York City (Art. 12-1) and today (1941) the use of the pneumatic caisson method is not so general as formerly.

In its details, the caisson for a building does not differ materially, except in the matter of size, from the bridge caisson. It is customary in most cases to use separate piers for all interior columns, these being circular or square in plan; but special conditions, such as the close spacing of two or more columns, or lack of clearance, sometimes make it necessary to use one pier for two or more columns. Where the grade of the cellar floor is below the ground-water line, the wall piers often serve the two following functions: (*a*) that of carrying the wall-column loads to rock and (*b*) that of acting as a dam or retaining wall to keep out the water. To accomplish the latter, they must form a continuous wall; hence they are made rectangular in plan, as wide as is necessary to give adequate working room and furnish the required stability as a dam or retaining wall—usually

between 6 and 8 ft.—and as long as can be conveniently handled, which is often as much as 30 ft. The ends of adjacent sections are then connected and made watertight.

For the circular form of caisson the diameter may vary from about 6 ft. as a minimum to 15 ft. or more. For a rectangular section the largest that has ever been used for building foundations is in the New York Telephone Building, where the largest caissons are 35 ft. 3 in. by 38 ft. 8 in. in plan. But more remarkable in many respects were some of the caissons used in the foundations of the Municipal Building, New York, one of which was 26 by 31 ft. and carried the load from five columns. In size this is not much larger than one used in the first building founded on pneumatic caissons, namely, the Manhattan Life Insurance Building, erected in 1893–1894, where the caissons had dimensions of 21 ft. 6 in. by 25 ft. 6 in. But in the magnitude of the single column loads and depth to which the caisson was sunk, a great development is apparent. The maximum load in the Manhattan Life Insurance Building was about 400,000 lb., whereas in the Municipal Building it was about 5,475,000 lb.; the depth of sinking below the street curb in the former was 54 ft., whereas in the latter it was 140 ft.; the maximum air pressures (gage) used were, respectively, 15 and 48 lb. per sq. in., the latter being within 2 lb. of the maximum allowed by state law. The 140-ft. depth below the curb corresponded to a depth of about 112 ft. below the level of general excavation. For the most part the depth to which pneumatic caissons for buildings have been sunk have ranged somewhere between 30 and 90 ft. below the curb, the true depth of sinking and the hydrostatic head worked against being less than this, depending on the amount of general excavating done before sinking the caissons and the position of the ground-water level, respectively.

11-2. Caissons of Timber. Caissons made entirely of wood have been extensively used in the past but are now of interest mainly from a historical standpoint. A typical timber pneumatic caisson is illustrated in Fig. 11-2*a*, which is one of the 12- by 24-ft. caissons used in the Gillender Building foundations. The sides of the working chamber were composed of two thicknesses of 12- by 12-in. timbers sheathed on the outside and inside with $3\frac{7}{8}$-in. material. The cutting-edge timber extended out beyond the walls, the outer part of the upper side abutting against the bottom of the outside sheathing, while the outside and bottom faces were protected by the cutting edge, which consisted of a steel angle and a vertical steel plate.

The roof consisted of three thicknesses of 12- by 12-in. timbers, the upper and lower ones running transversely, and the intermediate one longitudinally. The underside was sheathed with $3\frac{7}{8}$-in. material. The crib was composed of $3\frac{7}{8}$-in. sheathing, braced at intervals by horizontal frames of 8- by 12-in. timbers.

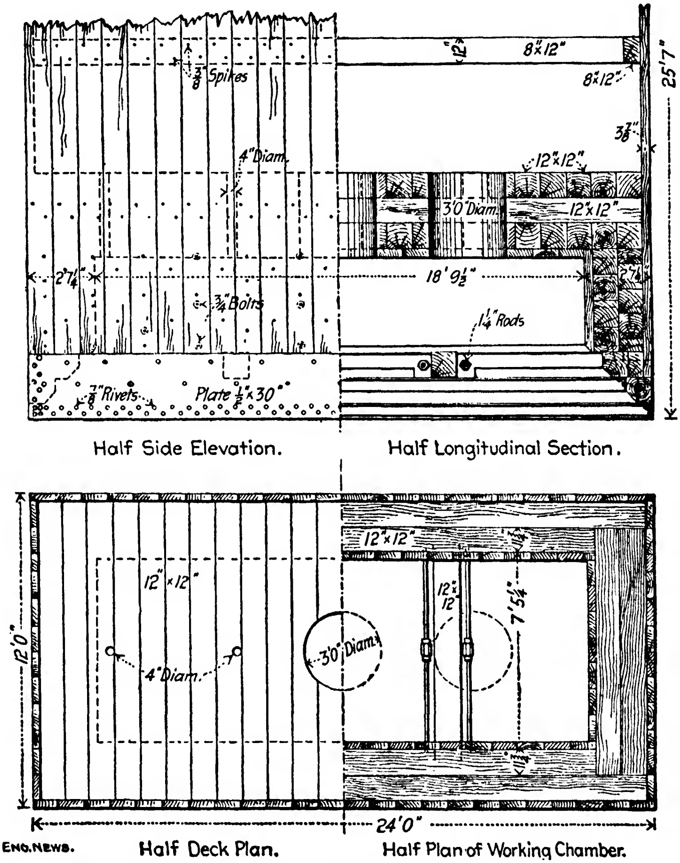
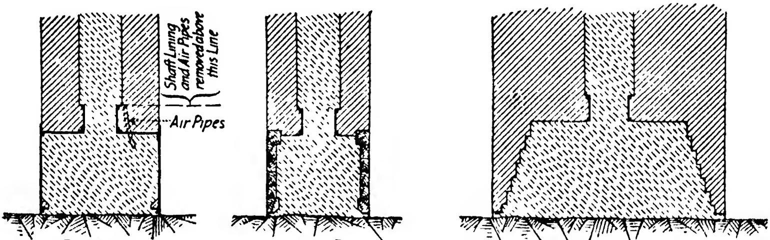


Fig. 11-2a.—Pneumatic Caisson of Timber Construction.

The foregoing example has permanent roof construction. The first wooden caissons doing away with this feature were those of the United States Express Company Building. Here the wall caissons were built with a width of $5\frac{1}{2}$ ft., a height of 6 ft., and lengths varying from 25 to 34 ft. The walls consisted of a single thickness of timber varying from 6 by 12 in. to 10 by 12 in. Across the 10- by 10-in. top course were placed 3- by 3- by $\frac{1}{4}$-in. angles running transversely and spaced 3 ft. apart, the vertical flanges

being turned up. The roof consisted of temporary sectional panels of $1\frac{7}{8}$-in. boards.

Instead of a crib, molds built of vertical tongue-and-grooved boards in sections 8 ft. high, having the same length and breadth as the caisson, were built on top of the latter to receive the concrete. They were held together by outside horizontal yokes, each made with 4- by 3- by $\frac{1}{4}$-in. angles forming a rectangular frame. Three yokes were used to a section, one at the top, one at the middle, and one at the bottom. When the forms were completed, a 6-in. layer of concrete was placed on the roof forms; and as soon as this had hardened somewhat, 2 ft. more of concrete was added. This $2\frac{1}{2}$-ft. thickness of concrete served as the permanent roof, the temporary wood panels underneath being taken off.



Type A.
Steel Working Chamber for s...all circular Caissons (6'6" to 8'6" Diam.)

Type B.
Timber Working Chamber for small rectangular Caissons (5'6" to 7'6" wide.)

Types C & D
Concrete Working Chamber for large circular & rectangular Caissons. (Circular 9' to 14'2"; rectangular 8' wide and over.)

Fig. 11-2*b*.—Types of Working Chambers, Municipal Building, New York.

Figure 11-2*b*, type *B*, shows the form of timber caissons used in the Municipal Building for rectangular caissons $5\frac{1}{2}$ to $7\frac{1}{2}$ ft. wide. It closely resembles those described in the two preceding paragraphs, the walls being made of a single thickness of 12- by 12-in. and 8- by 12-in. timbers, the bottom timbers being faced with 4- by 4-in. steel angles to form the cutting edge. It was provided with a temporary deck of 2-in. planks notched into the walls, which was removed after the first layer of roof concrete had hardened.

11-3. Caissons with Metal Shells. The advantages of the steel shell may be summarized as follows: (*a*) as the shell thickness is small, there is a maximum amount of working space in the air chamber, as well as a maximum amount of space to be filled with concrete; (*b*) for the cylindrical form it compares favorably in ease of construction with wood and concrete; and (*c*) it is easily made watertight.

The first pneumatic caissons used for a building, those of the Manhattan Life Building, were made of steel and were both circular

and rectangular in section. In making alterations to this building over 35 years after its construction, the structural steel in the caissons was found to be in excellent condition. Figures 11-3*a* and 11-3*b* show the details of both circular and rectangular metal caissons as used in the foundations of the Mutual Life Building. These caissons were sunk to solid rock, from 70 to 90 ft. below the curb and from 50 to 70 ft. below ground-water level. The roof of the cylindrical caisson was made of $\frac{9}{16}$-in. steel plates riveted to the lower flanges of 15-in. I-beams, as well as to the shell of the caisson. The latter consisted of $\frac{3}{8}$-in. steel plates braced at intervals with circular 4- by 4- by $\frac{1}{2}$-in. angles. The lower part of the shell, reinforced with an 18- by $\frac{3}{4}$-in. plate, formed the cutting edge. In the rectangular caisson the roof construction was very similar to that described above; but the sides were braced with brackets, extending from near the cutting edge to the roof as shown in the illustrations.

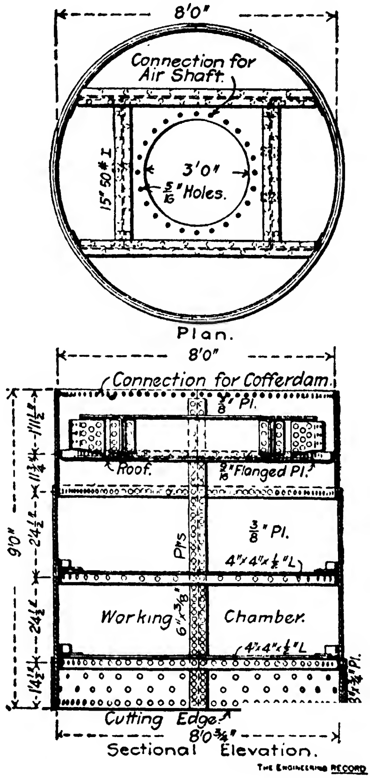


FIG. 11-3*a*.—Caisson for Inside Column. Foundation, Mutual Life Building, New York.

The $3\frac{1}{2}$- by $3\frac{1}{2}$- by $\frac{3}{8}$-in. angle riveted around the upper edge of the rectangular caisson formed a flange for the connection to the crib, which was built in sections 5 ft. high, of $\frac{3}{8}$-in. plate, with $3\frac{1}{2}$- by $3\frac{1}{2}$-in. angles for both top and bottom flange connections. Vertical 6- by 6-in. connection angles were used at the corners, and vertical $3\frac{1}{2}$- by $3\frac{1}{2}$-in. angles in the middle acted as stiffeners. For the circular caissons, butt instead of flange joints were used; here also $\frac{3}{8}$-inch plates in $6\frac{1}{4}$-ft. courses were employed for the crib. Both vertical and horizontal splices were butt-jointed with single splice plates on the outside.

Figure 11-2*b*, type *A*, shows the form of caisson used in the foundations of the Municipal Building, for all circular caissons having diameters between $6\frac{1}{2}$ and $8\frac{1}{2}$ ft. They were used in order to secure the maximum economy of space and material in the limited

area of the working chamber. The height of the working chamber was $6\frac{1}{2}$ ft., the walls being of $\frac{7}{16}$- and $\frac{1}{2}$-in. plates, shod with a 6- by $\frac{3}{8}$- or 4- by $\frac{1}{2}$-in. Z-bar cutting edge filled with concrete. The deck was formed of $\frac{3}{8}$-in. steel plates slightly domed, was flange-bolted to the walls and air shaft, and was removed after the concrete above it—which was placed before sinking commenced—had hardened, this being the same method as that already described for wooden

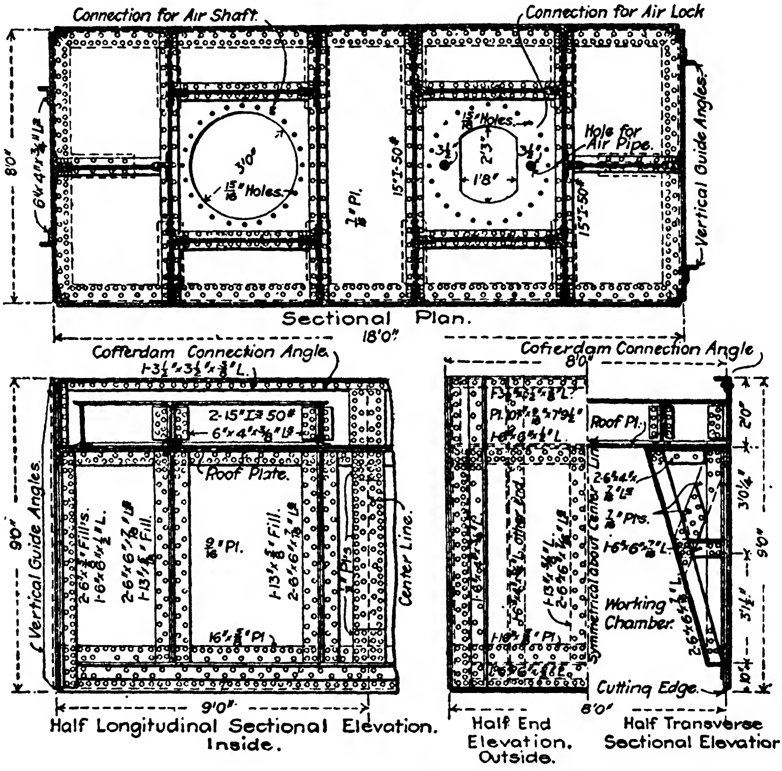


FIG. 11-3*b*.—Caisson for Wall Column, Mutual Life Building, New York.

caissons. The section of the air shaft shown remained a permanent part of the structure. The removable forms for the pier above the caisson were made of circular steel plate, held in place and reinforced with angle-iron rings, all of which are clearly shown in Fig. 11-3*c*.

11-4. Caissons of Wood and Steel. The caisson of wood and steel combined possesses some of the advantages of both steel and wood, but its origin is due largely to the fact that it was necessary to make a rush job in the pier sinking of a certain building, and for this reason only those shapes of structural steel were used that could

readily be obtained in the open market; the rest of the structure was made of wood, and the two materials combined in the simplest possible manner.

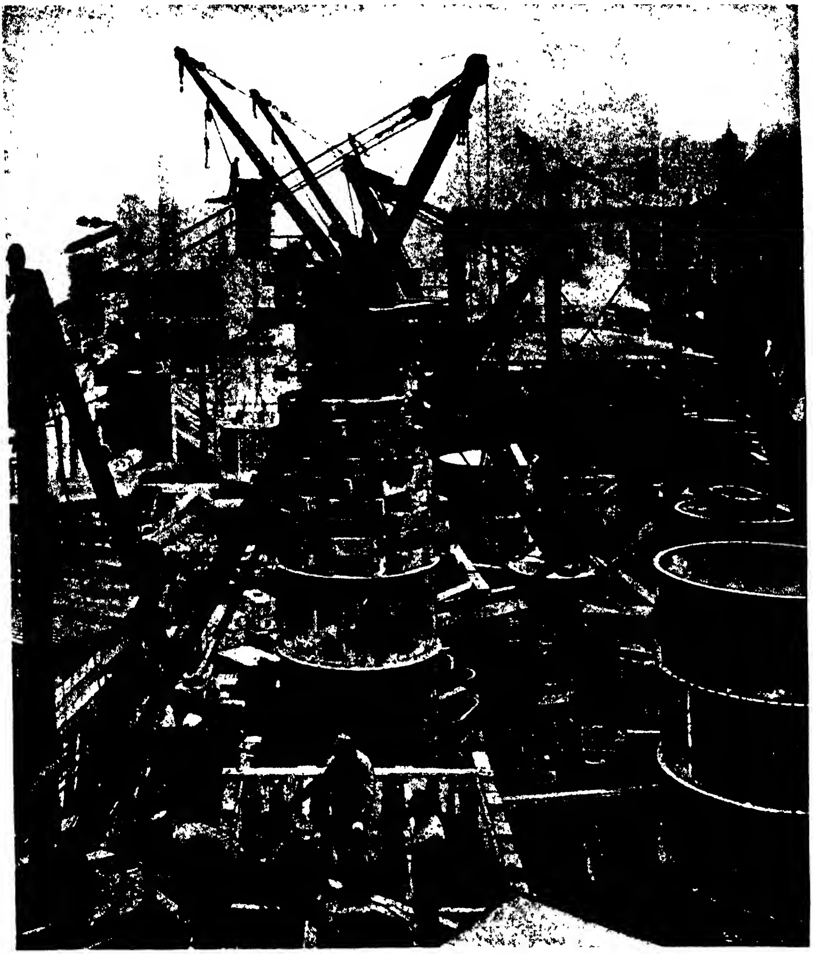


FIG. 11-3c.—Sinking and Concreting Caissons, with Steel Forms. Municipal Building.

For circular piers the caisson has a diameter varying from 6 to 12 ft.; for diameters less than 6 ft. it is difficult to excavate the material in the working chamber and, on the other hand, few single-column loads are large enough to require a caisson with a diameter of over 12 ft. The circular caisson is made of staves about 4 by 6 in. in section, usually dressed down to somewhat smaller dimen-

sions, the outer and inner surfaces being cylindrical. The staves are fastened, at every intersection, to inside 3- by 3-in. horizontal angle-iron rings, spaced from 3 to 5 ft. apart, bolts of about $\frac{3}{4}$ in. in diameter, countersunk into the wood, being used for this purpose. The staves are usually splined, but in some cases they are only calked. This type is illustrated in Fig. 11-4*a*.

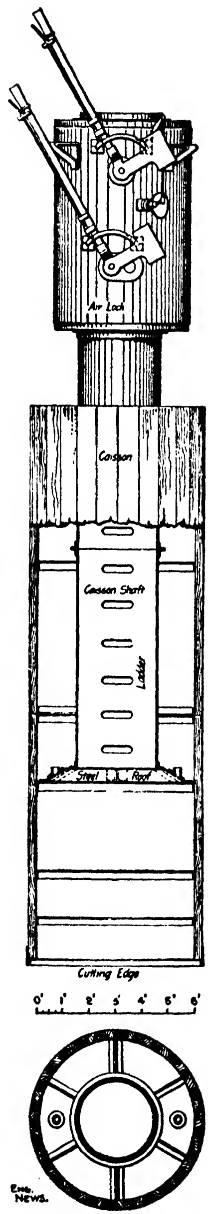


FIG. 11-4*a*.—Wooden Stave Caisson with Detachable Roof and Shaft.

In the circular caissons of the Atlantic Mutual Building, which had an average diameter of about 7 ft., the cutting edges were made with a 28- by $\frac{3}{8}$-in. steel plate. To give bearing surface to the cutting edge, in order better to control the sinking and to protect the feet of the staves, a 3- by 3- by $\frac{3}{8}$-in. angle was riveted to the inside of the plate, parallel to its bottom edge and $\frac{1}{2}$ in. above it, the horizontal leg forming a shelf to receive the lower ends of the staves. The roof of the working chamber was formed by a removable steel dome $\frac{1}{2}$ in. thick, made in two sections and stiffened with radial steel angles. It was calked with a hemp gasket and bolted to a 3- by 3-in. inside steel angle ring about $6\frac{1}{2}$ ft. above the cutting edge.

The crib was of the same form of construction as the caisson and was built as a continuation of the same to a height of 32 ft. above the cutting edge. Where the 32 ft. was not sufficient in height, short lengths were added on top. These were made in two semi-cylindrical sections, were butt-jointed to the top of the crib already in place, and were calked and bolted through the horizontal flanges of the angle-iron rings.

For the wall piers of the New York Stock Exchange Building the caissons were made rectangular in form, 8 ft. wide, from 24 to 30 ft. long, and 8 ft. high. They were sheathed with 4- by 12-in. vertical wooden staves with square calked edges and without splines. These staves were fastened to successive courses of inside horizontal steel angles, the latter extending wholly around the caisson. The longitudinal

walls were braced with horizontal transverse timbers resting on and bolted to the angle frames, as well as with tie rods, parallel and adjacent to the timber braces. The roof was formed with a removable steel-plate dome, reinforced with transverse angles and fastened to frame angles about 6 ft. above the cutting edge.

The crib was exactly like the caisson, except that it was without roof or cutting edge. It was built in sections 15 ft. high. The angle frames at the top of each section were set 3 in. below the top of the staves with the horizontal flange up. The angle at the bottom of the next upper section had its horizontal flange down and 1 in. below the lower end of the staves. This engaged the lower section and formed a tenon, thus binding the two sections together. A row of eyebolts, 1 ft. apart, connected the horizontal flanges of the angle frames.

11-5. Caissons of Reinforced Concrete. The chief advantages of this type of caisson are that it gives a monolithic pier, and that the caisson may be made at the site, thus avoiding the expense of hauling the same. One disadvantage is that the required thickness of walls so reduces the working space that this type cannot be used for very small caissons. Another disadvantage is the time element involved in waiting for the concrete shell to harden.

The foundation caissons of the Municipal Building were sunk in 1910 and were the first in which all-concrete caissons were used. Here both the circular and the rectangular forms were employed; all circular caissons having diameters 9 ft. or over and all rectangular ones having a width of 8 ft. or over were made of reinforced concrete. The right-hand illustration of Fig. 11-2*b* shows a section of the concrete caissons; it will be noticed that the walls thicken from the cutting edge to the roof by stepping the concrete. As noted in Art. 10-3, this is a better arrangement than the tapered form because it gives a positive bearing between the chamber shell and the concrete filling, thus making the whole area of the bottom available for carrying the load, without relying on any bond stress. The thickness of the bottom of the wall was about 10 in. and the real cutting edge consisted of a steel channel and a 4- by 4-in. steel angle, the former laid horizontally with flanges up and the latter with its vertical leg down, thus giving the sharp cutting edge and broad bearing surface. The walls were well reinforced with both vertical and horizontal rods.

No cribs were used, simple forms being employed in which to build a concrete shell, which was constructed before sinking was started. The caisson and the shell above the same were built

directly on the spot where they were to be sunk. The forms for the interior of both circular and rectangular caissons were made of wood, while for the exterior faces and for the shell above the roof they were made of steel for the circular ones and of wood for the rectangular ones. In the reinforced-concrete foundations for the Woolworth Building, New York, the inner forms were also of steel for the circular caissons.

11-6. Crib and Cofferdam. The frame which is built on top of the caisson and which, together with the roof of the caisson, virtually forms an open-box caisson, is generally called a cofferdam when applied to caisson construction for buildings. In the preceding articles, it was designated as a crib, since it corresponds to the crib of the bridge substructure. This frame is usually built in sections, as noted in the preceding pages, and the top section sometimes forms a true cofferdam. As water seldom covers the ground for such caissons, the cofferdam is not often employed; about the only time it is used is when the caisson is sunk before the general excavation for the cellar or subsurface floors is made. In the latter case the cofferdam serves as a form just as the crib proper does, but, after the general excavation is completed, the cofferdam is removed.

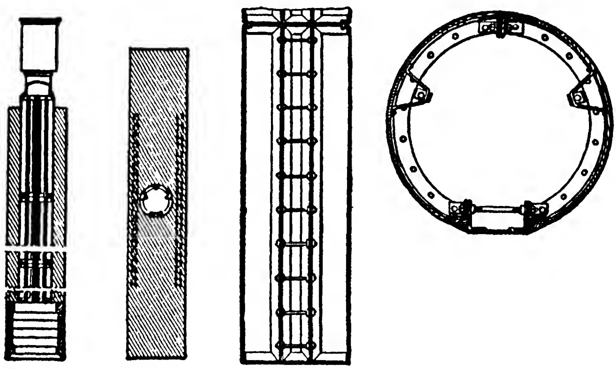
In the early deep foundations, such as those of the Manhattan Life Building, brick masonry was used for the pier material above the caisson, in which case the use of cribs was ordinarily dispensed with, the masonry being built up as the caisson sank. But this arrangement was not entirely satisfactory, for it was found that in omitting the crib the friction on the sides was much increased, which was a disadvantage in itself, and especially dangerous in that it tended to tear apart the brick masonry. Another desirable feature of the crib is that it enables sinking to be carried on without regard to the progress of the masonry construction.

When brick masonry was superseded by concrete, the latter being deposited on the deck of the caisson simultaneously with the sinking of the caisson or after it had reached rock, the crib became a necessity. Later the crib was eliminated by building a concrete shell—virtually the pier, except for the hole left for the shafts—before sinking operations were commenced.

If the caisson is not to be sunk over 30 ft., the entire length of shell is cast previous to any sinking, beyond that of pitching the caisson, that is, sinking the cutting edge a foot or two to give stability; however, if the depth is greater than 30 ft., the building and sinking are each done in two operations. This means that the pier is first built up part way, sunk till the top reaches the

surface of the ground, then the remainder built and the rest of the sinking done.

11-7. Shafts and Air Locks. Owing to the limited space in caissons used for building foundations, a single metal shaft usually serves for both men and materials. It is made removable as a matter of economy and also, as noted in Art. 11-1, to make possible a monolithic pier. One form of collapsible or removable shaft is shown in Figs. 11-7*a* *b*, *c*, and *d*, where *a* shows the shaft lining in place, *b* a plan of the caisson, *c* a vertical section of the shaft, and *d* a cross section of the same. Each section is composed of two approximately semicircular plates internally flanged for bolting to each other along one vertical edge and for a key interposed between the opposite edges of the plates. Horizontal internal flanges serve for

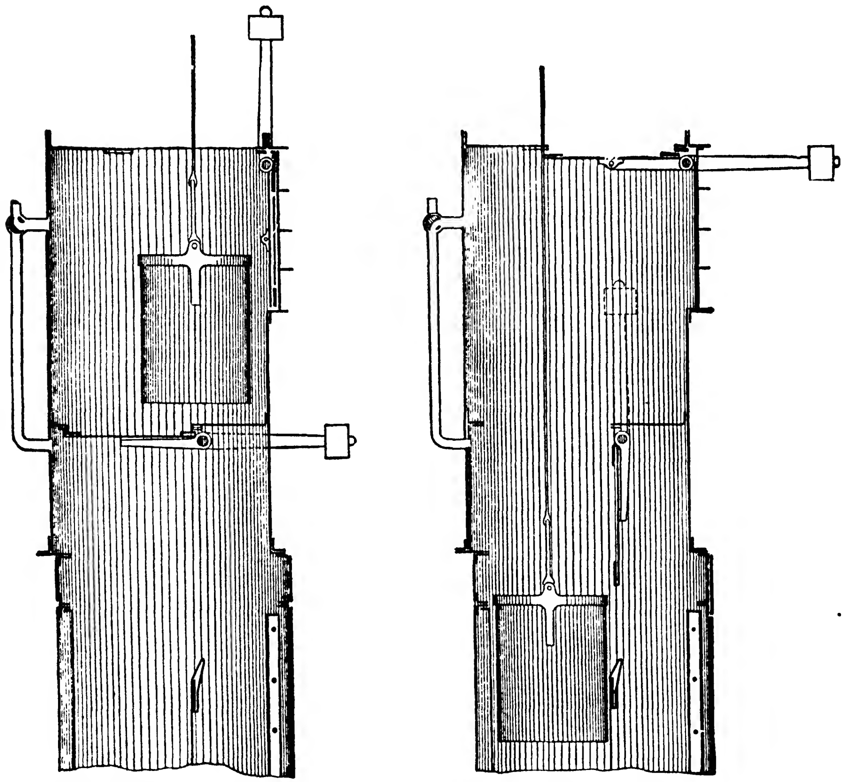


FIGS. 11-7*a*–*d*. Collapsible and Removable Shaft.

bolting successive sections to each other. Ladder rungs are arranged conveniently between the flanges of the key, and vertical guides are arranged just inside the line of the edge flanges to guide the bucket past them.

The shafts should be oiled or otherwise protected from adhering to the concrete. The bottom section of the shaft is usually not made removable but is thoroughly bonded to the concrete (see Fig. 11-2*b*). This is done to prevent the air in the working chamber from leaking between the crib and the air shaft. It also adds resistance against the tendency of the air to blow out the shaft and air lock. A somewhat better form of air shaft than the one just described has an elliptical section in which there is sufficient clearance for a man to pass between the bucket and the ladder, as shown in Fig. 11-7*e*. This lessens the danger to the men in the working chamber from the lodging of the bucket in the shaft.

The air locks are made of steel and are nearly always placed on the top of the shaft. Two types, the Moran and the Mattson—with some changes over the years (Art. 10-10)—have been used almost exclusively for work on building caissons. Figures 11-7*e* and 11-7*f* illustrate the Moran air lock used on the caissons of the Singer Building, New York. With the upper door open and the lower door



Figs. 11-7*e* and *f*.—Oval Shaft Arranged for Men to Pass Bucket. Moran Air Lock.

closed, the bucket is lowered into the air lock and moved to one side and the upper door closed, the rope passing through a hole in the doorframe. Air is then admitted to the lock from below; as soon as the prĕssure equals that in the shaft below, the lower door opens and the bucket is let down. The lower door remains open as long as the bucket is below. On coming out, the bucket is raised into the air lock, the lower door is closed, and the valve is turned to permit the air in the lock to flow to the open air. This results in the upper door opening, after which the bucket is removed from the lock.

11-8. Sinking the Caisson. Steel caissons are fabricated at bridge shops, assembled there or at the contractor's yards, brought to the site by trucks, placed in position by derricks, and sunk—this is the practice in New York City. The same general scheme is usually employed with caissons of wood, the main difference being that the material is fabricated in a woodworking mill, because of the limited space usually available at the site and of the lack of vacant lots in the near vicinity. As the average caisson with one section of crib seldom weighs over 10 tons, it is not a difficult matter to move them with trucks.

Before sinking the interior caissons, the site is usually excavated down to ground-water level; at least, this is true when the cellar floor is to be at or below that elevation. The caisson is then placed and one or more sections of the crib erected on the same, or a section of concrete shell is cast if no crib is to be used. The first few feet of sinking is accomplished without the use of air pressure. The material is usually dug by hand and removed with buckets, although the blowout process is occasionally employed. The disadvantage of the latter process is that the small volume of the working chamber makes it difficult to maintain a constant pressure in the caisson.

One of the gravest problems connected with sinking caissons for buildings is that of safeguarding adjacent buildings from undermining. When a caisson is sunk through quicksand within a few inches of a building, which, perhaps, is founded on a steel grillage, it is evident that great care must be taken not to disturb this quicksand under the grillage. This fact usually precludes the possibility of doing much "blowing," that is, suddenly reducing the air pressure in the working chamber to let the caisson sink a few feet, or using the water-jet on the outside to reduce friction.

On account of the large friction developed in sinking building caissons—much greater than with bridge caissons, where much of the crib is in water and therefore not subjected to friction—in addition to excavating the material from the caisson and filling the crib with concrete, special devices must be used to promote sinking. Greasing the sides of the caisson and crib reduces the friction somewhat, and it is usually advisable to do this. In some cases the caisson may be pulled down by attaching lines to caissons already sunk or to driven piles, as well as to timbers across the top of the caisson to be pulled down. By far the most effective and customary way is to weight the caisson temporarily with pig iron. Either heavy blocks, weighing as much as 4,000 lb. each, or ballast boxes filled with pig iron, are employed. A good example of the use of

large blocks may be seen in Fig. 11-3*c*. Some of the ballast boxes hold as much as 12,000 lb. of pig iron. The advantage of the blocks or boxes is that they require no special platform or yokes on the crib and are very quickly and easily placed and removed by the use of hoisting engines.

Some of the largest caissons sunk to considerable depths have each required as much as 1,000 tons of this weighting material, although the average caisson requires about 350 tons. From this it may be seen that for satisfactory cost a means of economically handling this weighting iron had to be developed.

In many of the earlier caissons, such as those of the Atlantic Mutual Building, described in Art. 11-4, an excessively large amount of temporary weighting was necessary, on account of the concrete not being placed until the caisson had reached its final position. This scheme was adopted in order that the roof of the caisson might be removed after sinking operations were over and the whole pier made a single monolith of concrete. But later caissons have preserved the latter feature without the expense of so much temporary weighting. As explained in Art. 11-2, this was brought about by using a thin temporary roof, only strong enough to hold 1 or 2 ft. of concrete on top.

At about the same time that concrete roofs came into use, cribs were largely dispensed with. In their place forms were used, and, as these forms were of light construction, the concrete was usually deposited in layers a few feet high and allowed to harden before more was added. As soon as the concrete was sufficiently strong, the forms were moved up and another layer of concrete placed. Where a considerable number of caissons are to be sunk, it has become standard practice to build the concrete as high as possible before starting to sink. The reasons for this are as follows: (*a*) sinking can be done at a much more rapid rate than can the building of the concrete, (*b*) it saves on the number of times that pig iron must be loaded and unloaded, and (*c*) it makes less temporary weighting necessary.

Another advantage where the caisson is to serve also as a dam is that it helps eliminate leakage and percolation. Experience has shown that it is almost impossible to make a joint that will not leak.

In the caissons for the City Investing Building the concreting was entirely finished before excavating in the working chamber was commenced, although in some cases caissons were sunk a few feet to give lateral stability to the tall shafts and to relieve

the excessive weight on the walls of the working chamber. In the Singer Building, where bedrock was 70 ft. below the surface, the concrete was built on the caissons to one-half the estimated total height before sinking was started, after which the caissons were sunk until the top of the concrete was down to the surface of the ground, when the sinking operations were stopped. The remainder of the concrete was built and sinking resumed. In some of the piers of the Municipal Building three build-ups were necessary, the maximum height of any one build being 60 ft.

With these high piers great care is necessary in guiding them while sinking. For the caissons of the United States Express Building heavy horizontal frames, braced with inclined struts, inclosed them. These frames took bearing on greased vertical guide strips attached to the faces of the concrete after the forms were removed.

11-9. Rate of Sinking. Although showing large variations, the average rate of sinking caissons in New York City is high, largely because rush jobs are customary there. Because of the high value of real estate, owners are willing to pay well for keeping the time required for placing the foundations down to a minimum. For this reason many of the records in sinking were not made under natural conditions, the cost being considerably higher than if more time had been taken.

The caissons of the Manhattan Life Building, which were both circular and rectangular in plan, the former shape averaging about 12 ft. in diameter and the rectangular shape about 320 sq. ft. in ground plan, were sunk a distance of 34 ft., mostly through fine sand. This sinking was done, the cribs filled with masonry and the working chamber and shafts filled with concrete, on an average of one caisson in 8 days. This corresponds to a sinking rate of $4\frac{1}{2}$ ft. per day.

The caissons for the Atlantic Mutual Building (Art. 11-4) did not have their cribs filled with concrete until sinking was completed. The material penetrated was largely quicksand. One caisson was sunk 24 ft. in 7 hr. Forty-two caissons were sunk and concreted in 36 days.

In the caissons of the Trinity Building the average rate of sinking through soft material such as quicksand was about 1 ft. per hr., whereas through hardpan it was only about one-third as much. These caissons were very similar to those of the United States Express Building (Art. 11-2) and were built up previously to sinking. The rate as here given refers only to the actual sinking

and not to the time spent in building up the caisson, filling the working chamber, etc.

What is probably the best record ever made in caisson sinking was the placing of 87 caissons, all over 75 ft. in depth, in 60 days. These were placed for the foundations of the Trinity Annex and United States Realty Buildings by the Foundation Company of New York City.

11-10. Filling the Air Chamber. Where the caisson is to rest on rock, the surface should be thoroughly cleaned of loose and friable material before placing the concrete filling. If hardpan, without any pockets of loose material in it, overlies the bedrock, it is rarely advisable to carry the cutting edge of the caisson more than a few feet into the hardpan. The best method is to stop sinking the caisson at hardpan level and to carry the excavation below the cutting edge through the hard material down to rock. In this case, when the concrete is placed, it will bond to the hardpan and so reduce the load on the base, whereas, if the caisson is sunk through the hardpan to solid rock, this bonding effect is lost. Another advantage of stopping the caisson at hardpan lies in the ease with which the bottom section may be belled out to distribute the load over an area larger than the horizontal section of the caisson.

For caissons in which the roofs are to be removed on the completion of the sinking, the working chamber is filled with concrete, which is allowed to harden for about 2 days. The removal of the roof and shafts follows, and the remaining space is filled with concrete.

11-11. Watertight Dam of Wall Piers. In New York, the high value of real estate results in the building of deep cellars, as many as three floors below street level now being used. Three floors below ground level for the building of the First National Bank of Jersey City resulted in an increased floor area of 25,000 sq. ft., the foundation work costing less than if individual caissons had been built under the 80 odd columns, which would have been necessary with the lowest floor at street level. These deep cellars demand heavy dam construction around the sides from ground-water level to cellar level. As explained in Art. 11-1, this dam construction is obtained by making the wall caissons rectangular in plan and sinking them with a small clearance between the ends of adjacent piers; afterward the space between the piers is filled with concrete or clay to form a continuous and watertight dam. Some clearance must be left in order to allow for slight deviations in sinking, the usual amount allowed being from 4 to 18 in. The space between the

piers may be made watertight down to bedrock, in which case the use of the pneumatic-caisson process may sometimes be avoided for the interior column piers, or the space may be made watertight to a level a little below the level of the cellar floor.

The first building using this form of dam construction was the Commercial Cable Building. The clearance between the caissons varied from 4 to 10 in. As soon as the caissons were sunk, 3-in. pipes were jetted down in the space between the end walls, and clay pellets were forced through these pipes into the sand by means of a plunger operated by a pile driver. As clay filled the space, the pipes were gradually raised until the surface was reached, thus forming a watertight dam of clay. As soon as this was completed, a section of the metal shell in the middle of the ends was removed, the material excavated, and the open space filled with concrete, semicircular wells having been left in the middle of the ends of the caisson.

The caissons for the wall piers of the New York Stock Exchange Building were sunk with a clearance of less than 2 in., the average being 1 in. That this is too small a clearance was demonstrated in this work. Watertightness was obtained in the following manner: As the crib and working chamber were filled with concrete, semicircular wells were left in the ends. On the completion of sinking, the adjoining wooden walls were drawn together and bolted[1] as shown in Fig. 11-11*c*. The central part of the walls was then removed, thus combining the two wells into one, which was filled with concrete to bond the two piers together.

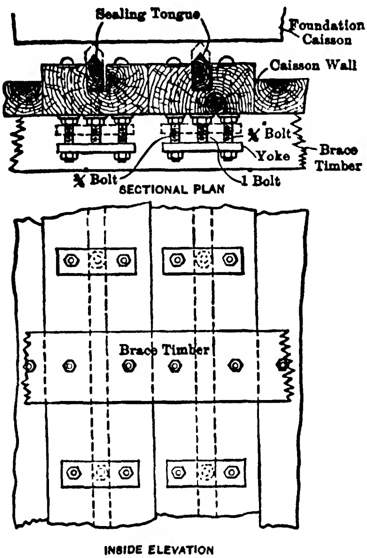
The spaces between the caissons of the Bank of the State of New York Building were sealed by using two 2-in. vertical strips of timber on alternate caissons. These strips were recessed into the wall as shown in Fig. 11-11*a*. On the completion of sinking, the strips were forced out against the adjacent caisson by the simple arrangement shown in the illustration.

The method used for connecting the caissons for No. 42 Broadway is illustrated in Fig. 11-11*b*. When the sinking was completed, the sand between the guide timbers was removed by jetting the same, and the space was filled with grout.

The method used in the piers of the Trust Company of America Building, where a 12-in. clearance was used, is illustrated in Fig. 11-11*d*. As shown in section *XX*, semioctagonal spaces in the center of the ends of the piers were left as wells when the concrete shells above the caissons were built, this building being done pre-

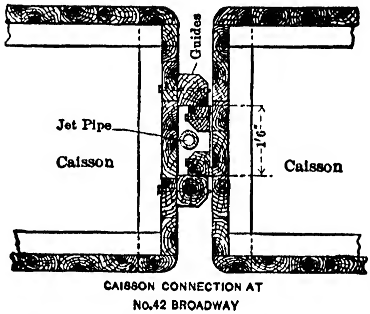
[1] From Recent Developments in Pneumatic Foundations for Buildings, by D. A. Usina, *Trans.* A.S.C.E., vol. 61, December, 1908.

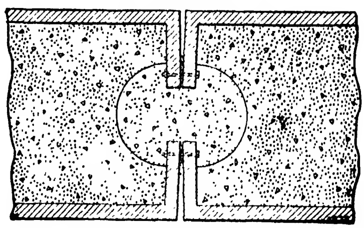
vious to the sinking. After the caissons were sunk, the earth in the 12-in. space between the cores was excavated to a depth of 1 ft., and the upper boards AA were removed, cut, and placed in the position A'. This alternate excavating and sheeting was carried down a few feet, after which the core planks were removed and a short section of a steel air-shaft cylinder set into it and concreted; the air lock was placed on top. The slots S were filled with the



CAISSON CONNECTIONS,
BANK OF STATE OF NEW YORK.

FIG. 11-11*a*.





CAISSON CONNECTION,
STOCK EXCHANGE.

FIGS. 11-11*b* and *c*.

shaft concrete and acted as keys to prevent the blowing out of the shafts. Air pressure was put on, the remainder of the material was excavated, boards were placed down to the top of the caisson, and the whole chamber was filled with concrete.

A very neat arrangement was used in the caissons for the United States Express Building, where there were clearances of from 6 to 12 in.[1]

Vertical grooves about 2 ft. wide and 8 in. deep were made in the ends of the wall piers and formed, with the clearances already noted for the caissons, wells from 22 to 28 in. wide above the tops of the working chambers. Compound sheet piles were made with 3-in. planks wide enough to overlap the corners of adjacent piers at each joint and were driven close

[1] *Eng. Rec.*, vol. 53, p. 316, Mar. 3, 1906.

to the inner and outer faces of the piers so as to cover the joints between them. . . .

After the sheet piles were driven, 4-in. pipes were jetted down in the corners between their edges and the outer faces of the piers, and as they were withdrawn grout was forced through them, which effectually sealed

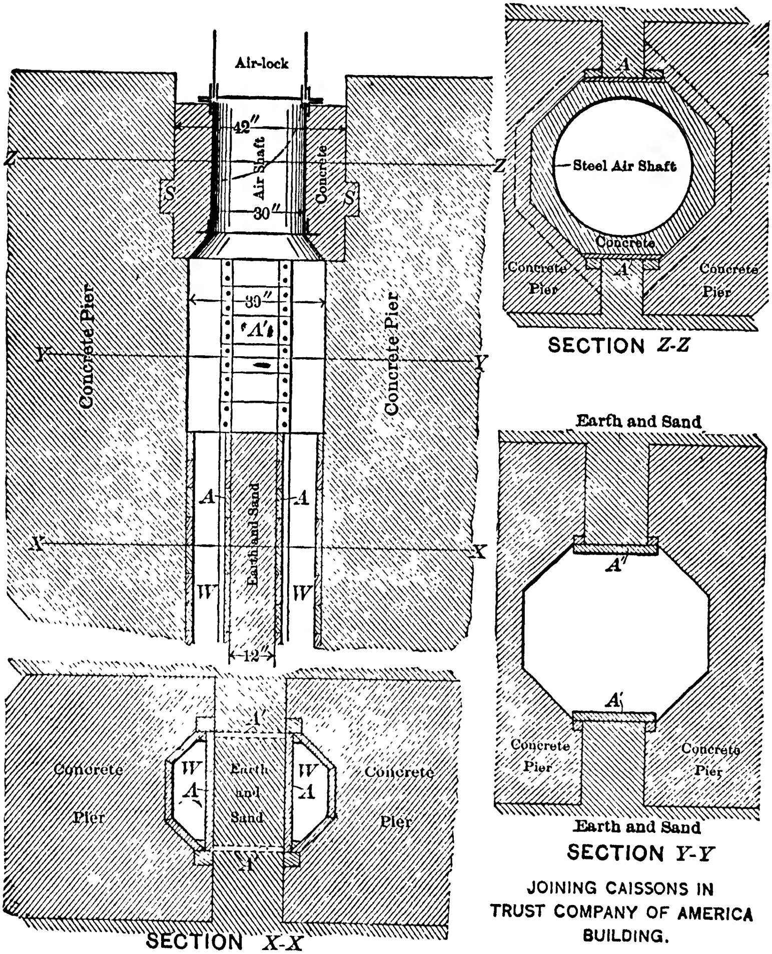


FIG. 11-11*d*.—Method of Sinking Joint Well between Caissons.

the spaces between the piles and the piers. Men were then able to enter the well between the ends of the piers and excavate the quicksand and hardpan down to the tops of the caisson, calking as they went any slight leaks between the sheet piles and the piers. Jet pipes from 2 to 6 in. in diameter were sunk in the narrow space between the caissons and removed or loosened the material down to the cutting edges. Grout was then

introduced through them and, with the sand and broken stone already there, formed concrete, thoroughly sealing the space between the working chambers. Afterward the well above the working chamber was rammed full of ordinary concrete, thus making a solid key which united the wall piers and prevented leakage.

Figure 11-11*e* shows a line of wall column piers and the form of bracing usually employed. The whole area of the building is first

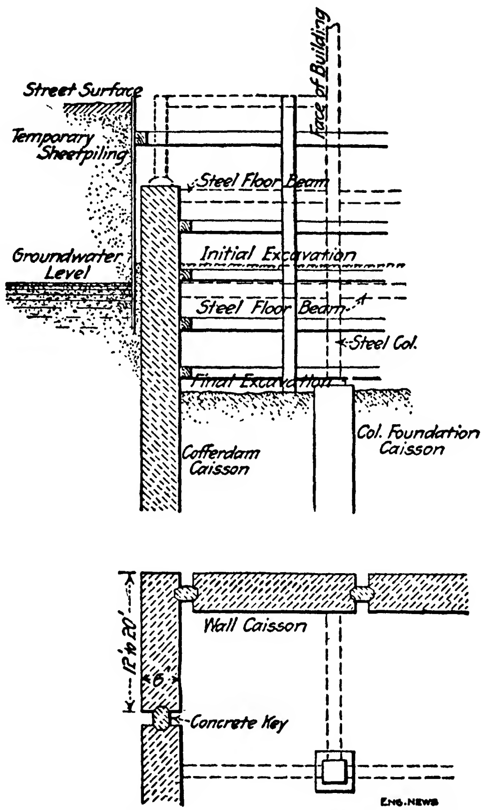


FIG. 11-11*e*.—Connection and Bracing of Wall Caissons.

excavated to ground-water level and sheeted, after which the wall piers are sunk and keyed. The interior is excavated to cellar-floor level, the wall piers being temporarily braced as the excavation proceeds. The final bracing of these piers is done by means of the floor beams of the building.

Figure 11-11*f* illustrates one of the most satisfactory types of joints. Two 6- by 8-in. timbers are attached to the caisson to be sunk first and extend the entire height of the caisson. Following

the sinking of the caissons, a 4-in. steel pipe is jetted down in the space between these timbers, and, after the sand has been washed out of the pipe, clay cylinders are rammed down and forced out of the pipe by means of a ramrod, gentle blows from a pile driver assisting the process. As the ramming progresses, the pipe is raised at intervals until the clay seal is completed to the top. Care must

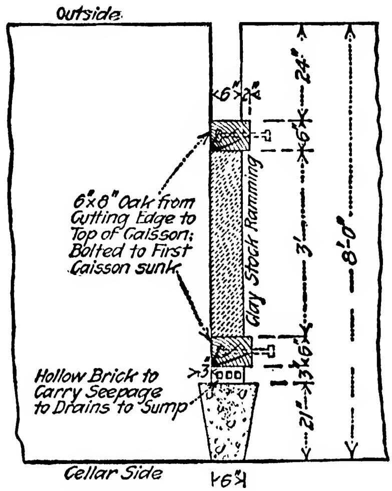


FIG. 11-11*f*.—Watertight Joint between Caissons.

be exercised not to overdo the ramming, else the caissons may be forced apart.

This type of joint has successfully withstood a head of water of 35 ft., while cellar excavation was being carried on as well as the excavation of the sand in the joint as far back as the first timber separator. The latter space was filled with concrete after a layer of hollow brick had been placed to take care of any seepage through the clay fill. According to T. K. Thomson, this type of joint costs only about one-seventh that of the compressed-air joint, illustrated in Fig. 11-11*d*.

CHAPTER XII

LAND FOUNDATIONS IN OPEN EXCAVATION AND CONTROL OF WATER

12-1. Predraining Foundations. In placing building foundations, the problem of controlling water is not generally so serious as in bridge foundation work. This has resulted in the development of methods of placing land foundations quite different from those described in the preceding chapters. In some cases the whole site is excavated to foundation level, whereas in other cases a series of cylindrical shafts are sunk. Where water-bearing sand is present, predrainage methods may be used. In clay, water is often not a problem.

Predrainage consists in lowering the ground-water level by pumping the water out of the soil to any desired elevation and maintaining it at that elevation. The well-point method of predrainage dates back to 1838, when Robert Stephenson used it in building a tunnel in England. In this country it was used as early as 1889 in the construction of the No. 2 Ridgewood pumping station in Brooklyn, N. Y. However, it was not until 1927 that general recognition was given its inherent advantages through the experience gained in building the foundations for the Harriman Building in New York City. Prior to this date it was believed that foundation piers could be safely sunk through the water-bearing sands of lower Manhattan only by the pneumatic process (Chap. XI). Now, by predraining the sand, foundations are placed safely, quickly, and cheaply by means of either open-pier wells or full-lot excavation.

In predraining work, well-point units consisting of vertical pipes and well points are sunk, usually by the jetting method, through or around the site to be drained, these pipes being coupled to horizontal header pipes hooked up in turn to portable pumping units.

In the design of the well point—which consists essentially of a jetting nozzle, brass-mesh screening, valve shell, and valves—the problem is to provide valves by which, in sinking the pipe, the water pumped into the pipe will flow out at the base of the well point and, when suction is applied, the water will come into the well point through the side screens. This is accomplished in the patented

Moretrench design, as shown in Fig. 12-1*a*. There are two valves, a ring valve and a ball valve. When the jetting water comes down through the center pipe, it passes through the center of the ring valve and drives the floating ball valve off its seat (from its full-line to its dotted-line position), thus opening up a channel for the downward movement of the water. As the water leaves through the bottom of the tip, it encounters earth resistance and the pressure

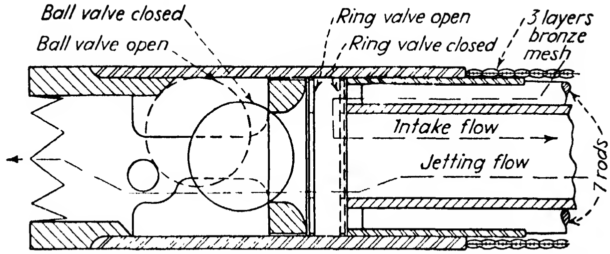


FIG. 12-1*a*.—Well Point. (*Courtesy of Moretrench Corp.*)

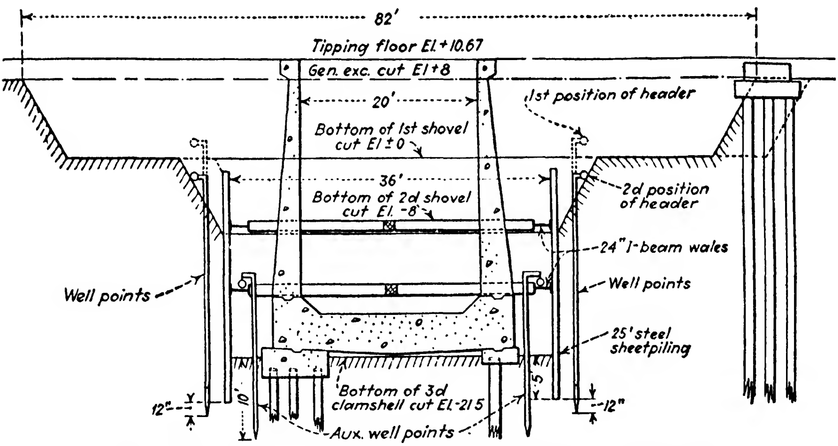


FIG. 12-1*b*.—Section of Bin Pit Showing Well-point Arrangement.

partly flows back along the ribs and out through the screen. This movement picks up the ring valve off its seat (full-line position) and positions it as shown by the dotted lines, where it effectually stops the water escaping through the screen. On the other hand, when suction is applied, the wooden ball valve closes, while the ring valve remains open, thus causing all the entering water to come in through the screen.

The well-point method of predraining is illustrated in Fig. 12-1*b*, where it was desired to construct in a water-bearing fine-sand foundation a deep receiving bin, 20 by 151 ft., for a destructor plant

in New York City. Ground-water level was at elevation 0. The entire plant site was first excavated to elevation +8, after which excavation was further carried to ground-water level. A set of well points were then installed around the four sides just outside the sheet piling to be later placed. The spacing of these well points was 3 ft. along the sides and 6 ft. along the ends. The pipe units were composed of $1\frac{1}{2}$-in. pipe 22 ft. long with a 3-ft. screened point at the lower end. The header circuit consisted of 8-in. pipe, to which two pumps were connected.

Pumping lowered the water sufficiently to enable another 8-ft. shovel cut to be made. The sheet piling was then driven and braced

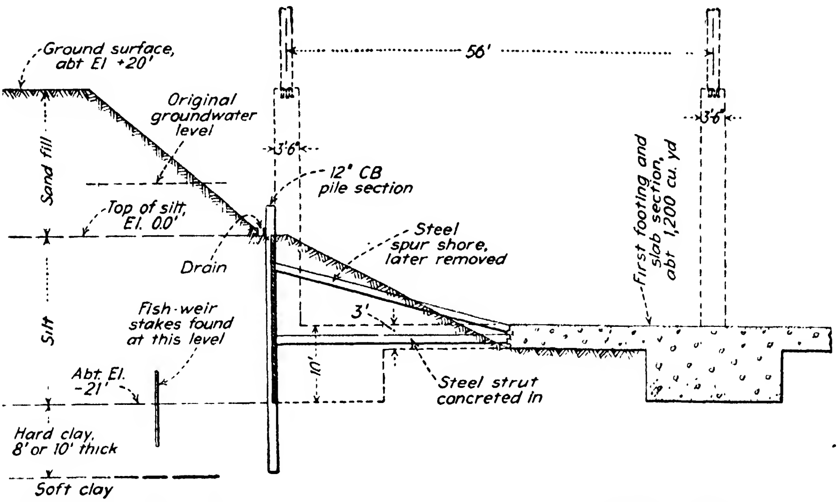


FIG. 12-1c.—Foundation Construction Procedure for New England Mutual Life Insurance Co. Building in Boston.

with 24-in. I-beam wales and a horizontal set of timbering placed on the bottom of the cut. The well points were jetted down another 3 ft. and the excavation continued for 6 ft., after which another set of bracing was installed. As the first set of well points were inadequate to keep the ground dry, a second set was installed inside the sheet piling, these being spaced at 6-ft. intervals. When excavation was completed, the foundation piles were driven, and a concrete mat was poured.

In placing well points for the construction of a 28-story addition to the New York Telephone Company Building in New York City in 1931, difficulty was experienced in getting the well points down due to the presence of gravel. Finally a 6-in. pipe with a conical cast-iron point was driven at the location of each well point. After

inserting the well-point unit in the pipe, the latter was withdrawn leaving the cast-iron point and well-point unit in the ground. After dewatering, the material was removed and the foundation of the new building placed. This consisted of a reinforced-concrete mat 6 to 7 ft. thick bearing on a layer of coarse sand. The underside of this mat was 35 ft. below street level and 20 ft. below ground-water level.

In placing the rigid-frame foundation for the office building in Boston described in Art. 13-11, the work was done in an open excavation without any well-point pumping. The soil consisted of 20 ft. of sand fill, overlying 20 ft. of silt, which in turn rested on clay. A test pit sunk to subgrade showed that the sand drained easily and that the silt was practically impervious. The sand was first excavated down to the silt, with sides sloping back as shown in Fig. 12-1*c*. A clay-lined wooden drainage gutter was built at the foot of this slope to intercept the ground water draining from the sand. A series of 12-in. H-sections were then driven at 8-ft. spacing just inside this gutter, after which the silt was excavated dry, with sides sloped and trimmed to 40 deg. Excavation was then made for the interior wall footing, and the footing was poured to serve as a support for the diagonal spur shores used to brace the H-sections as the silt was excavated to a vertical face. During excavation, horizontal lagging was placed back of the flanges of the posts. Later on, horizontal struts were placed between the footing block and the H-sections.

12-2. Open Wells with Sheeting: The Chicago Method. Predrainage is not applicable to clay soils owing to their impermeability or resistance to the flow of water. However, in many instances this impermeability is a decided asset, making it possible to place foundation piers in open excavation with only a moderate amount of pumping. This is the condition that obtains quite generally in Chicago and which has led to the development of the so-called "Chicago method."

In the business district of that city, for a distance of about 14 ft. below the street curb, the soil consists of loam and made ground. Below this there is a layer of clay having a thickness of from 70 to 80 ft., which overlies hardpan or gravel and solid rock. The upper 6 to 12 ft. of this clay is hard and stiff and forms the bed on which rest many steel-grillage foundations (Chap. XIII) which, dating from 1878, were so extensively used in that city. Below this the clay becomes softer and remains so down to hardpan. In general this softer clay differs from that above only in the larger

amount of water carried. In places, pockets of quicksand are present in the soft clay.

The clay is stiff enough to permit sinking wells in sections about 4 ft. deep, each section being sheeted with 2- by 6-in. or 3- by 6-in. planks in 4-ft. lengths as soon as a section is excavated. Sometimes the sheeting is made of reinforced concrete or of sheet metal. The wells vary from 3 to 12 ft. in diameter.

The wells are usually excavated by hand, from one to four men working in a single well. As soon as the sheeting is placed, two or three hoops, composed of bars (often $\frac{3}{4}$ by 3 in.), angles, or channels, are placed in position to brace the sheeting of each section. These hoops are made in semicircular shape with their ends bent inward to form flanges that are bolted together. As soon as the bracing of one section is placed, the next section is excavated, and the lagging for that section is placed abutting against the lagging for the section above.

Care must be exercised to fit the lagging tightly against the clay to prevent the starting of flow. Another form of hoop designed for that purpose consists of four lengths of T's, bent to form a circular rib bearing against the lagging, and of a hollow central hub to which are attached jackscrews radiating from the hub like the spokes of a wheel. The heads of these jackscrews are fitted to shoes on the horizontal web of the circular rib or rim. As many jacks as necessary may be used, but not less than four, one for each section of the rib. The jacks may be set up to compress the surrounding material as much as desired.

The spoil is removed from the wells by buckets operated by a windlass or other arrangement. For small jobs the windlass may be worked by hand, but, where a large number of piers are to be sunk, power is used. The lagging and bracing are sometimes removed as the concrete filling is placed, but, if the surrounding clay is at all soft, they are usually left in. The concrete may be as rich as a 1:1:2 mixture or as lean as a 1:3:5 mixture, depending on the load to be superimposed.

Where the pier rests on hardpan, the lower part is ordinarily belled out to about twice the diameter of the pier, the belling being done at an angle of about 45 deg. The bearing pressure commonly allowed in the Chicago hardpan is about 7 tons per sq. ft., and on solid rock 30 tons per sq. ft.

12-3. Applications of the Chicago Method. The first building in Chicago and the first in the United States, with the exception of the City Hall of Kansas City, to have this type of foundation was

the Chicago Stock Exchange, built in 1892. This structure was founded on piles and on piers sunk by the Chicago method, the latter being used where it was feared that the jarring of pile driving would disturb the foundations of adjacent buildings.

The foundations for the new City Hall of Chicago were composed of circular concrete piers from 4 to 10 ft. in diameter and seated on bedrock 96 to 120 ft. below street grade. Three-inch tongue-and-grooved lagging in 4-ft. lengths was used. The clay spoil was dug by hand, one to four men working in a well at one time. The buckets held 3 or 4 cu. ft. and were raised and lowered by means of timber tripods set up over the wells. A drive wheel was placed on one side of each tripod and was connected to a shaft that carried a winding spool. A single endless cable on a hoisting engine connected with a number of the driving wheels—the tripods being set up in straight rows—and thus readily served seven or eight wells.

The deepest foundations ever placed by the Chicago method are those for the Cleveland Union Terminal Building, where the base of some of the tower piers are 243 ft. below street level and the depth of well reaches a maximum value of 204 ft., the difference between these figures representing general excavation. The maximum pier load was 8,900,000 lb., the diameter of the pier being 10 ft. 4 in. A concrete mix of 1:1:2 was used, with a unit stress of 750 lb. on concrete not reinforced and 1,000 lb. under the steel billets of the column bases, where the pier top was reinforced by hooping and vertical bars.

The wells were started with square pits lined with 3-in. horizontal crib planks down to ground-water level. Below this and extending through the sand stratum and into the clay, steel sheet piling was used.

Starting at the level of the clay, the Chicago method was used, the lagging consisting of tongue-and-groove maple, cut 6 in. wide and 6 ft. long. For the largest wells the lagging was 3 in. thick and each set of lagging was braced against the sides of the holes by two steel rings 1 by 4 in. in section. It was found that the strength of the rings depended largely on the character of the bend of the lug. With too much radius at the bend there was a tendency for the ends to roll in. To force the lagging tight against the sides of the wells, steel split wedges were driven between the lugs at the ends of the semicircles. Where the rings proved inadequate, drum braces were installed. As shown in Fig. 12-3*a*, these drums were of two built-up segments composed of four to six layers of 2-in.

planking formed to fit the circumference of the hole. The two segments were placed opposite each other and were braced apart with 8- by 8-in. timbers and screw jacks.

For a depth of 100 ft. electric winches and tripod rigs were used to remove the excavated material in 4-cu. ft. buckets, while for greater depths electric hoists operating 1-yd. buckets by means of standard head frames were used. The best record of excavation was 16 cu. yd. for one shift, there being four workmen in the well. The average rate was 4.53 ft. of depth per shift.

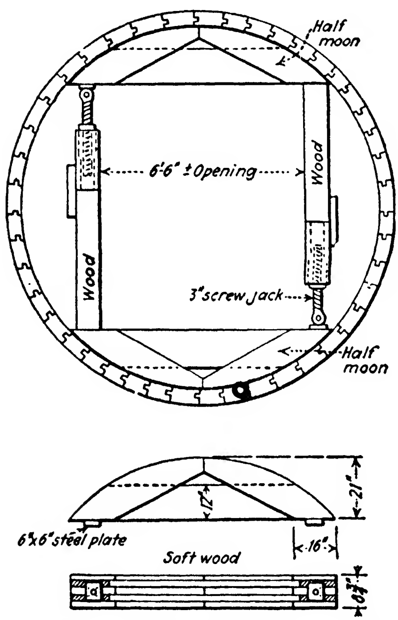


FIG. 12-3a.—Typical Details of Bracing Drums.

For one of the foundations of the Field Building in Chicago precast concrete lagging was used, the cross section of the lagging being T-shaped, with the stem placed inward and the flanges bearing against the clay. The edges of the flanges were tongue-and-grooved to interlock, and the stems were beveled, with the thinnest part near the flange, to dovetail into the later-placed concrete of the pier. The flanges were $5\frac{1}{2}$ in. wide and $1\frac{1}{2}$ in. thick, the stem 2 in. long and 2 in. thick at the inside, tapering to $1\frac{1}{2}$ in. at the flange. Reinforcing consisted of welded fabric in the flange and a $\frac{3}{8}$-in. bar in the stem.

In another foundation job in Chicago where the characteristic clay appeared at a depth of 26 ft., a layer of fine saturated sand overlay this clay. Here steel shells, 4 to 6 ft. in diameter and 30 ft. long were jetted through the sand and sealed 4 ft. in the clay, the sealing being done by driving with a steam-hammer. Six or eight 2-in. water pipes were carried down inside the shell and terminated in $\frac{1}{2}$-in. nozzles. Each pipe carried one jet directed downward in line with the inside face of the shell and another jet directed upward along the outside surface.

12-4. Modifications of the Chicago Method. The clay is often too soft to permit excavation before placing the lining. Under these conditions the Chicago method must be modified.

Figure 12-4*a* shows the method used in placing piers for a power station in Detroit, where there was 86 ft. of soft material followed

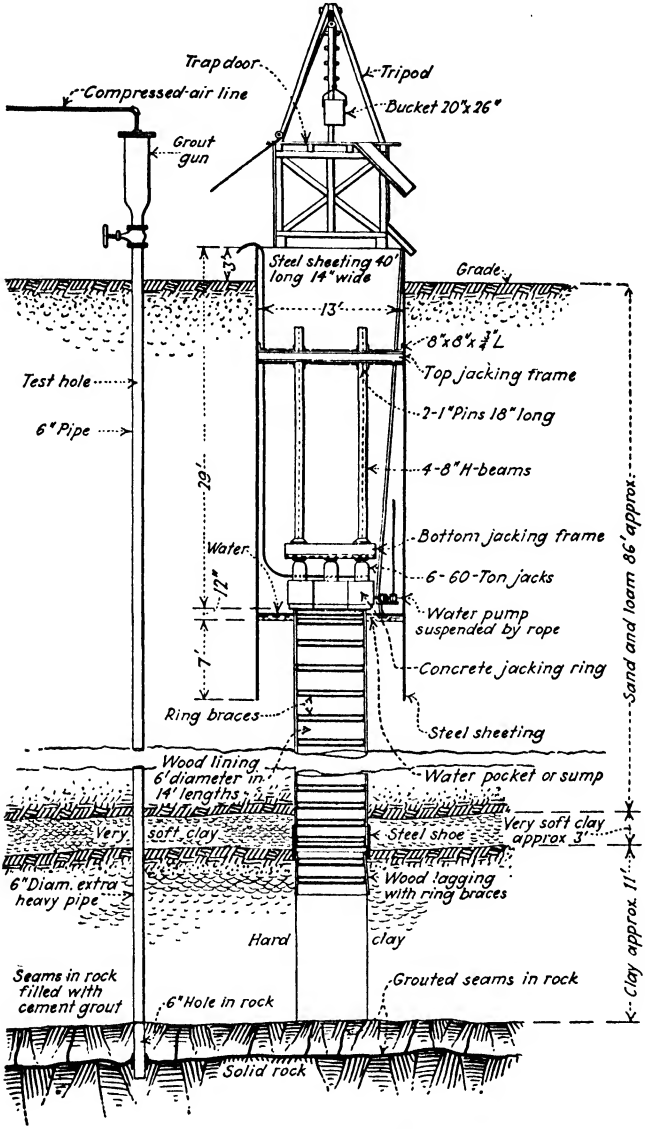


FIG. 12-4*a*.—Jacking Wood Lining for a 6-ft. Shaft for a Power Plant in Detroit.

by 3 ft. of very soft clay and then 11 ft. of stiff clay to bedrock. For about 33 ft. the well was 13 ft. in diameter and was lined with interlocking steel sheet piling in 40 ft. lengths, braced with annular

rings. Below this was a 6-ft. well extending to rock, which was filled with concrete to form a foundation pier. The lagging for the 6-ft. well was of 3- by 6-in. tongue-and-grooved pine, made up into cylinders 14 ft. long, held in shape by interior ring braces of 6-in. channels. The shaft could not be excavated in advance of the lagging, and so these cylinders were built complete on the outside and then lowered into place and jacked down, top lengths being added as the work progressed. A steel shoe $2\frac{1}{2}$ ft. high, made of $\frac{3}{8}$-in. plate reinforced with a ring brace of a 10-in. channel placed 2 in. above the cutting edge, was placed on the bottom.

Sometimes metal shells are used in which case the sections are about 8 ft. long and telescope one into another, each section being about 2 in. smaller in diameter than the one above it. A section is first driven, after which the material is excavated, and the next section driven. After the hole has been sunk to the required depth, the concrete filling is placed, the sections of shell being removed at the same time. In firmer clay the sections may be placed after excavation has been made.

A 10- to 12-ft. layer of quicksand with its surface 100 ft. below the street curb was struck in sinking the wells of the Chicago Edison Company's building. Below this there was a layer of boulders overlying the bedrock, and these boulders varied from cobblestone size to 5 ft. in diameter. On reaching quicksand, the Chicago method was abandoned and steel cylinders in three sections, with vertical joints flanged with angle-iron connections, were sunk. As the quicksand was removed from the interior, these cylinders sank by their own weight until the boulders were reached. The boulders had to be drilled and split open to permit the steel cylinders, aided by jacks, to pass through.

In the wells for the foundations of the Northwestern Railroad Terminal[1] the pneumatic-caisson process was used when a heavy water-bearing stratum just above rock was struck.

12-5. Open Wells with Sheet Piling. Where there are alternating layers of sand and clay, it may not be possible to predrain the soil because of the presence of clay or to use the Chicago method because of the freely flowing water in the sand. Under these conditions wells lined with sheet piling are used, the piling being driven in one or more sections. The upper sections have diameters large enough to permit offsetting and placing the lower sections inside.

The upper section of piling is driven first; this may be done by hand or by machine. If the driving is not difficult, it is done before

[1] See *Eng. News*, vol. 62, p. 554, Nov. 18, 1909.

excavating is commenced, since there is less likelihood of the surrounding material being disturbed by flowing into the well from underneath the piling. Care should be taken to start the sheet piling in its correct position, as this will save much trouble later. On excavating the wells, which is commonly done by men with picks and shovels, throwing the spoil into buckets lowered into the wells, bracing should immediately be placed.

As soon as the first section is driven and the material excavated, the second section is started. On the completion of the work to hardpan or rock, the bottom is carefully cleaned and leveled and the lower section filled with concrete, a 1:2:4 or 1:3:5 mixture being used and the sheet piling serving as a form. The latter may be withdrawn after the concrete has set, or it may be left permanently in place. If the sheet piling is to be withdrawn, the concrete should be protected in some manner from bonding to it. Above the lower section special forms are usually made for the pier and the whole, including the sheet piling, withdrawn after the concrete has set.

Figure 12-5*a* illustrates the cylindrical well sunk to rock for the 22-story Railway Exchange Building, St. Louis, Mo. The illustration indicates the character of the material through which it was sunk, as well as the distance sunk. For the upper section, 8 ft. in diameter, 2- by 6-in. tongue-and-grooved wooden sheet piling was used. It was driven by hand and braced by 3- by $\frac{3}{4}$-in. two-piece rings. The lower section had a smaller diameter and was composed of 9-in. Lackawanna steel sheet piling, braced with four-piece wooden drums made of 12- by 14-in. material with cast-iron ball-and-socket joints at the ends. After the piling was driven, the material, loosened and kept in suspension by a $1\frac{1}{2}$-in. jet under 100 lb. pressure, was removed with pumps. On the completion of the excavation, a 3-ft. layer of $1:1\frac{1}{2}:2$ concrete was deposited to seal the bottom, no pumping being done in the meantime. After allowing the concrete to set for 5 or 6 hr., the water was pumped out and the lower cylinder filled with concrete, the braces being removed at the same time. The sheet piling was left in place.

In placing cylinders varying from 33 to 81 in. in diameter and 35 ft. below ground surface for a building in Tonawanda, N. Y., a timber pile was first driven on the center line of each pier. The material around the pile was then excavated to water level and a templet set in position. A timber tower about 35 ft. high was built and an upper templet, 22 ft. above the lower one, placed and held in position by the tower. A 14- by 14-in. timber mast was then

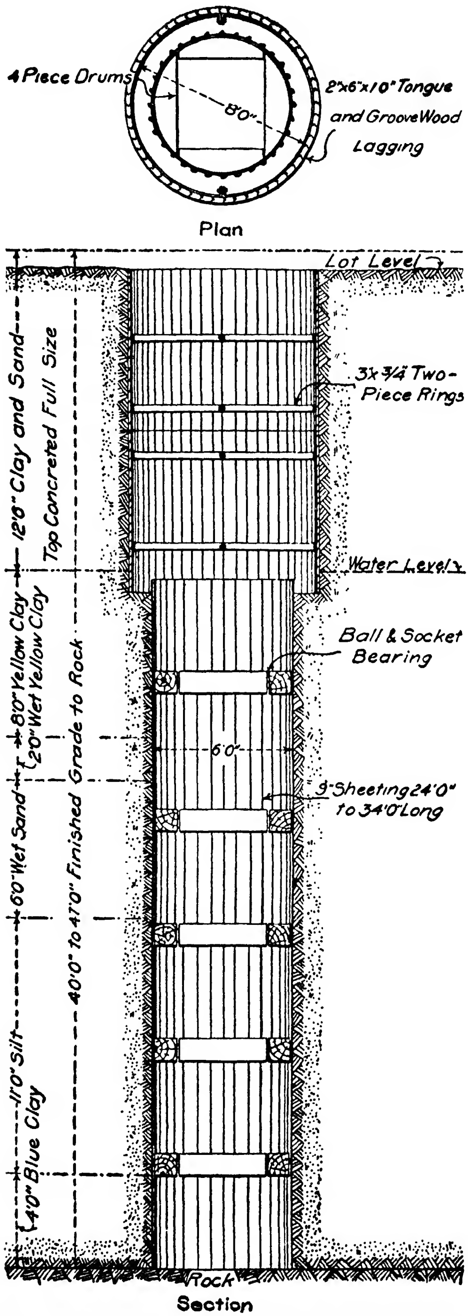


Fig. 12-5a.—Open Well for Railway Exchange Building, St. Louis.

mounted and held in position on top of the wooden pile by a 2-in. steel pin, and it was guyed at the top.

All the steel sheet piling was placed in position and interlocked by means of a cableway with two timber portable towers 70 ft. in height, the position of the cableway permitting all the piles in one cylinder and all the cylinders in one row to be assembled without moving the tower. A steam pile-driving hammer was then supported from the mast by a steel A-frame, the hammer being free to slide up and down and the mast free to revolve. Driving was then started, two piles being driven 3 or 4 ft. at a time and then the next two, and so on until all the piles reached bedrock.

The material in the cylinders consisted of mud, silt, sand and, near the bottom, clay containing quantities of glacial drift. This material was removed by a sand-jet pump consisting of a 40-ft. length of 6-in. pipe with an elbow at the top to which was connected a horizontal 10-ft. length of pipe. Running down the 6-in. pipe on the outside were two jet pipes with reducers at the end. One jet pipe was turned upward at the lower end and toward the center of the 6-in. pipe, while the other was straight and extended about 1 ft. below the bottom of the 6-in. pipe. A third jet was placed at the elbow of the large pipe. The latter pipe was then placed on the material in the cylinder and the jets turned on after closing a gate valve at the elbow of the 6-in. pipe. This forced all the water downward, resulting in the rapid sinking of the pipe. After it reached bottom, the valve was opened and in less than 2 hr. all the sand and loose material had been removed.

When excavation was completed, a cage of reinforcement was placed in the cylinders, and the cylinders were filled with concrete by the tremie method.

12-6. Use of Boring Machines. The use of boring machines for well-excavation work is one of the more recent developments in foundation practice. Two general methods may be employed, the dry or the wet. In the dry process a rotary excavating machine or suspended auger is used, the details of which are quite variable, but they all embody the general principle of a cutting machine or auger supported on a rotating vertical shaft, powered by electric motors.

In one make of machine the cutting device consists of a split circular pan having a pronounced pitch and fitted with two sets of cutting teeth. This pan is mounted on the lower end of a vertical pipe shaft. At the upper end of the shaft there is a motor, usually from 20 to 40 hp., and a reduction gear to furnish power for rotating

the shaft. The whole assembly hangs from the load line of a long-boom crawler crane. Figure 12-6*a* shows this type of excavator mounted on the rear of a truck.

Another type of boring machine is carried on a frame attached to a power shovel and consists of a cylindrical bucket at the lower end of a vertical shaft. The bottom of the bucket is equipped with cutting edges, whereas at the top of the bucket there are adjustable-extension reaming knives, which enables the machine to cut holes of various diameters, as well as to undercut lagging in place in the upper part of the hole. The vertical shaft consists of telescoping

FIG. 12-6*a*.—Excavator Mounted on the Rear of a Truck.

sections by which the bucket is lowered and raised. The bucket is so designed that it is closed while boring and then the bottom opens for discharging the excavated material.

Holes up to 8 ft. in diameter and over 120 ft. deep may be drilled by this method. In placing the foundation of the Detroit Post Office in firm clay, 106 wells, having an average diameter of 6 ft. and an average depth of 118 ft., were dug by a single machine in 60 working days, the total excavation amounting to 13,400 cu. yd. The average rate per day was 208 ft. It was first thought that a depth of about 40 ft. could probably be bored before lagging the wells, but it was found that the holes would stand open all the way down to hardpan. The lagging was built up in sections and dropped down the hole.

Where the soil is lacking in stability or carries a large amount of water, the wet process may be used. This consists of first liquefying the soil in the space to be occupied by the pier by means of a boring machine. Figure 12-6*b* illustrates one type of drill used. It consists of a hub with arms radiating out, cutting or scarifying teeth being mounted on these arms. This hub is attached to the lower end of a hollow drill stem 10 or 12 in. in diameter. The drill stem is rotated from power introduced at the top and water under heavy pressure is forced down and out of the hub, emerging at the point to assist the cutting action and to aid in emulsifying the soil cut away by the drill. The overflow is carried to a sump and then recirculated. By this method the well is kept filled with a fluid of considerably greater specific gravity than water, which aids in holding the sides of the hole.

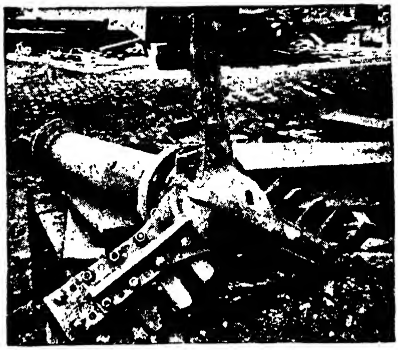


FIG. 12-6*b*.—Spud Drill Used in Boring by Wet Process. (*Courtesy of Raymond Concrete Pile Co.*)

The usual procedure after completion of drilling is to lower a steel cylinder into the mud-laden hole and drive it to a final seal in the bottom; or, if on rock, it may be sealed by using a sand-cement mixture around the outside of the shell at the bottom. After this the semiliquid mass is removed by pumping or by excavating with a clamshell or orange-peel bucket. The bottom is then cleaned off and concrete poured in the usual way. Under some conditions the cylinder is withdrawn as the concrete is placed.

As an experiment, in placing some 40 of the 200 piers required for the foundation of a Montgomery Ward Building in Chicago, the concrete was placed by the tremie method without using any lining or without pumping out the slurry.

12-7. The Grouting Process. The general idea of the grouting process is to inject fluid cement between and among materials already in place and thus cement the mass into a solid concrete. The process may be used for forming new foundations or for repairing old ones. For foundations on land two general methods may be used: The whole foundation bed, down to rock or other firm material, may be turned into concrete *in situ* and the piers built directly upon it; or a ring of concrete may be formed around the site, forming a sort of cofferdam, after which the interior may be

excavated down to solid material and the substructure built within it. The latter method will give a more reliable foundation but a more expensive one. In using the former method, it is a difficult matter to prevent pockets of uncemented material from being present. It seems that this process may be used satisfactorily for any material varying from the size of broken stone down to fine sand. Clayey material cannot be grouted.

Where cement grout is used in fine material, two pipes, a short distance apart, are first driven. Water is then pumped down one pipe and in taking a course of least resistance will come up the other pipe, thus cutting out a channel between the two pipes. By using a number of pipes, as many channels as desired may be made. As soon as a well-developed channel is formed, cement grout is pumped through the pipe instead of water. When the grout appears in the outlet pipe, the latter is closed by a valve and the pumping continued, thus forcing the grout to permeate the sand around the channel. In this way a stratum of solid mortar or concrete is formed; by employing the same scheme at various depths, the whole mass becomes solidified.

Where medium-sized material is encountered, it is often only necessary to drive a row of pipes and pour the grout into them, the head being sufficient to force the grout throughout the material.

In coarse material the difficulty lies in keeping the grout within bounds and preventing it from spreading out in thin layers and running into adjacent territory, or below the level at which it is desired to form the concrete. This difficulty may be overcome by using the principle of successive accretions. In using this method, only a small amount of grout is poured into any one pipe at a time. After this has had time to set, more grout is poured in and the operation repeated until a solid floor of concrete is made. Walls may be made in the same manner, after which the interior may be filled with grout or excavated.

Grouting has been used quite extensively for repairing dams, quay walls, etc., where the water has washed out the filling. In such cases it is customary to sink pipes and pour cement grout into them, the pressure head on the grout being sufficient to force it into place. It has been found that the pressure head of a column of grout is about double that of water. For an example of the use of the grouting process for the foundation of a cylinder caisson, see Art. 10-8. In Art. 8-9 attention is called to the application of the grouting process for stopping leakage in a cofferdam.

In Detroit the bedrock, which is as much as 125 ft. below street level is badly fissured and contains water high in sulfur compounds. In placing cylinder foundations for buildings, it is customary to grout these fissures in order to better control the flow of water and also because of the injurious effect of the water on concrete. In building the 80- by 270-ft. Union Trust Building, twenty-three 6-in. holes were put down with a well driller, the casings being driven to bedrock and the holes drilled into the rock until a flow of sulfur water was encountered. Cement grout was then forced into the crevices in the rock under air pressures as high as 100 lb. per sq. in. One of the holes took as many as 800 sacks of cement, and a total of 3,570 sacks was used for all the holes. In Fig. 12-4*a* is shown a grouting apparatus for another building foundation in Detroit.

12-8. François Cementation Process. By the use of the François cementation process a special grout pump, capable of exerting pressures up to 3,000 lb. per sq. in., forces cement grout through an armored hose attached to a special head on a pipe about 2 in. in diameter. Using a high pressure makes it possible to force the liquid through soil of considerable density.

This method was used in consolidating the soil under the footings of a reinforced-concrete viaduct in Charleston, W. Va., where the settlement of the foundations prevented the new structure from being opened to traffic. The footings of each pier, designed to take a load of 2 tons per sq. ft., were 9 ft. square and extended about 9 ft. below ground surface. They rested on filled-in material that extended, on the average, 10 ft. below footings. Beneath the fill were layers of sand, gravel, and clay, with bedrock from 40 to 50 ft. below ground level.

Hollow sectional drill rods were forced down to rock by compressed-air hammer drills. The $1\frac{3}{4}$-in. diameter drill bit had several openings along its length, and during the drilling a small amount of water was passed down the rods to keep the holes clear. The injection of cement grout was commenced with the drill bit at full depth, using François ram-type grout pumps for the purpose. As resistance to injection increased by the filling of the voids at that depth, the rods were slowly raised to allow the ground around the hole to be impregnated to full depth.

Commencing from the abutment end, early work showed that there was an exceptionally large percentage of voids, particularly around 20 ft. below surface; ground water evidently had carried a large amount of finer material into a near-by culvert. As this made it impossible to prevent the

grout from spreading beyond the footing that was being injected, a series of vertical holes at 6-ft. centers was bored around each pair of piers from 5 to 6 ft. outside the edges of the footings. Into these holes, sand mixed with a small amount of cement grout, was forced. This procedure blocked up the washed-out channels and helped to prevent the grout subsequently injected from spreading away from the footings. For the injection of neat cement the drill rods were forced down at a slight inclination from the vertical, in order that the grout should be introduced into the ground immediately under the footings. Each grout hole was injected up to a pressure of 200 lb. per sq. in., with grout varying from one part cement to three parts water, up to equal proportions, according to the nature of the ground encountered. Enough grout holes were put down to obtain a uniform consolidation, as shown by hardness and resistance to drilling further holes.[1]

12-9. Chemical Soil Solidification.[2] Another method of solidifying quicksand layers is by the use of chemicals. Chemical soil solidification was developed in Europe and has been used there with success for a number of years. A number of processes have been developed, the Joosten Process perhaps being the best known.

In this process two chemicals are used, the first being predominantly a commercial brand of silicate of soda, diluted to the required specific gravity. The second chemical is a strong solution of calcium chloride. The first chemical is injected into the ground using injection pipes of special design. These pipes are similar to well points, with rows of perforations near the lower end. The pumping is done in stages as the injection pipes are driven. The horizontal spacing of the pipes depends on the resistance to flow as in the case of the grouting process.

The second chemical is injected as the pipes are withdrawn in the reverse order of stages. The two chemicals immediately react on each other to form a silicic gel, which fills the voids of the loose sand and binds the grains together in a solid mass. Tests indicate a compressive strength of solidified material of from 280 to 1,280 lb. per sq. in. Field-test solidifications made in Germany showed no diminution of strength, compactness, or impermeability after a period of 10 years. Under average conditions, cost of this process is said to be from $20 to $30 per cubic yard of solidified material.

12-10. The Freezing Process. The idea of freezing the soil, as an aid to excavation, has existed for many years, and, although it has attained a considerable degree of success in the sinking of

[1] See *Eng. News-Record*, vol. 107, p. 299, Aug. 20, 1931.

[2] Material of this article contributed for the most part by C. Martin Riedel.

mine shafts, particularly in Germany and other foreign countries, it has seldom been applied to foundations. However, owing to the inherent possibilities of this process for foundations at great depths, the principles are worthy of careful study.

The presence of water causes the principal difficulties in foundation work, especially when water is present in very fine sand, forming what is known as quicksand. If the water can be frozen, the work becomes easy. In the method invented in 1884 by F. H. Poetsch, M. D., a Prussian, tubes are driven around the outside of, or into, the soil, all over the site to be excavated, and a freezing mixture is made to circulate through these pipes, which gradually transforms the soil into a nonwater-carrying solid mass, after which the excavation can easily be made. If the pipes are driven to a nonwater-bearing stratum, it is only necessary to freeze a wall around the site, but, if an impervious stratum is not reached, the whole site, or a ring around the site and a layer of soil near the bottom, must be frozen.

Long watertight tubes closed at the bottom, from 4 to 6 in. in diameter and spaced about 3 ft. apart, are first driven through the mass to be frozen. Inside of these tubes are placed small pipes, from 1 to 2 in. in diameter, which are open at the bottom or have openings in their sides near the bottom. A considerable number of the small circulating tubes are joined together by a larger pipe, and the larger or freezing tubes are capped and joined together by another pipe. A circuit is then formed and cold brine is drawn from a tank, pumped down the circulating tubes, up through the freezing tubes, and back to the freezing machine. For shaft sinking, the pipes are usually placed around the circumference of a ring with perhaps a few inside which are so insulated that they freeze only the bottom of the shaft.

What is said to be the first application of this process to building foundations is that for the substructure of a department store in Berlin. Figure 12-10*a* illustrates the conditions obtaining at the site as well as the general plan of the process. The subsoil was a quicksand with ground-water level about 13 ft. below the curb. The foundations of adjoining buildings were 10 ft. below the curb, while the excavation for the new structure had to be carried to a depth of 36 ft. below the curb. Sheet piling was first used, but, as soon as the excavation reached below waterlevel, the sand from under the adjoining buildings on one side of the lot commenced boiling up in the excavation, causing several structures to settle and crack. The freezing process was then adopted. Freezing

pipes 5 in. in diameter and about $\frac{5}{16}$ in. thick were sunk on 3-ft. centers, as shown in the illustration, and extended 59 ft. below curb level. The circulating pipes were 1 in. in diameter and were connected to a supply header at the top, while the 5-in. pipes were connected to a drain header. The liquor passed through the circulating pipes with a velocity of $11\frac{1}{2}$ ft. per min. About 4 weeks after the brine was started, the ground was frozen a sufficient dis-

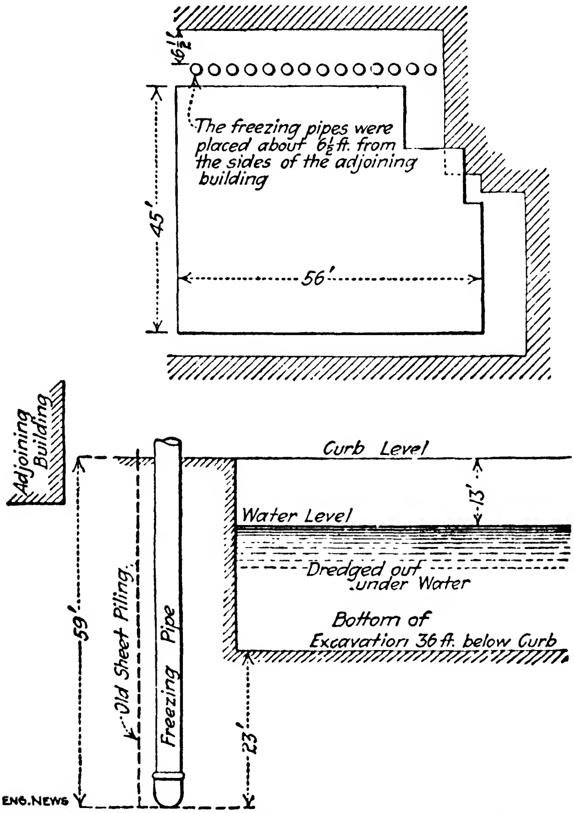


FIG. 12-10a.—Building Foundation Constructed under the Freezing Process.

tance to begin excavating, after the completion of which the foundation was placed. The cost is said to have been lower than if the pneumatic-caisson process had been employed.

To construct ventilating shafts for a tunnel at Antwerp, Belgium, through 85 ft. of fine saturated sand to a clay bearing, 116 steel tubes were placed for each shaft, these being on the circumferences of two circles, the diameters of which were 86 and 78 ft. The circumferential spacing was $4\frac{1}{2}$ ft. Inside these 6-in. tubes 2-in. pipes

were placed. After properly joining up the system, a calcium chloride brine solution was circulated at a temperature of from −10 to −20°F. Four months were required to freeze a wall from 10 to 15 ft. thick, inside of which the excavation and construction work was successfully carried on.

The freezing process was successfully applied to a leak in the cofferdam for the west river pier of the Detroit-Superior arch bridge over the Cuyahoga River at Cleveland. The head of 46 ft. caused the steel sheet piling to bulge badly, which resulted in the leak. The cost of stopping the leak by this method was about $1,600. Before using the freezing process, about $10,000 had been expended in trying to stop the flow of water by other methods.

Only under special circumstances, or where no other process can be adopted, or where a refrigerating plant is located near by, will the freezing process prove commercially practicable.

CHAPTER XIII

SPREAD FOUNDATIONS

13-1. Historical. Foundations for heavy buildings, where rock is some distance below cellar-floor level, are of three general types, two of which—pile and caisson or well—have been discussed in foregoing chapters. The third type is the shallow or spread foundation. The object of this type is to spread the load over a considerable horizontal area near the surface. A pile foundation either carries the load immediately down to solid material by point bearing or else distributes it out by side friction through the adjacent soil mass. By using caissons or open wells, the load is transferred directly to bedrock.

Years ago it was general practice to make shallow foundations of heavy buildings one continuous bed of concrete or solid masonry. For example, the foundation bed of the Government Post Office and Custom House in Chicago, built in 1877, was a solid mass of concrete 3½ ft. thick. Owing to the wide variations in the several column and wall loads, as well as to the low moment resisting capacity of the unreinforced concrete, the building settled so much—about 24 in.—and so unevenly that it had to be replaced after a service of only 18 years.

Another early type of spread foundation was that of the inverted masonry arch which was first used in the Drexel Building, Philadelphia, built in 1893, the details of which are shown in Fig. 13-1*a*. Another notable example of the use of this type was in the World Building, New York City. Both of these structures were among the early examples of the modern steel office building. In the Drexel Building the arches were made of brick which distributed the column loads to the soil below through continuous lines of concrete bases in the column rows.

The shallow type of foundation is relatively inexpensive and easily and quickly constructed, but it does not furnish a rigid and unyielding support. Where founded on compact sand, the settlement will be slight, seldom over ½ in., but, where founded on material like the Chicago clay (see Art. 12-2), the settlement may in time reach 2 ft. or more. Hence heavy buildings resting on shallow

foundations are built to allow for some settlement, or else are so constructed that powerful hydraulic jacks can be used to raise the building to permit shimming up. Uniform settlement is generally not troublesome, but unequal settlement causes the walls to crack and may cause difficulty in operating elevators, etc.

The most satisfactory method of guarding against differential settlement was found to be the use of independent footings for the columns, the area of the base of each footing being proportioned to give equal unit bearing pressures under all footings. This method was first advocated by Frederick Baumann in a pamphlet entitled "The Method of Constructing Foundations on Isolated Piers,"

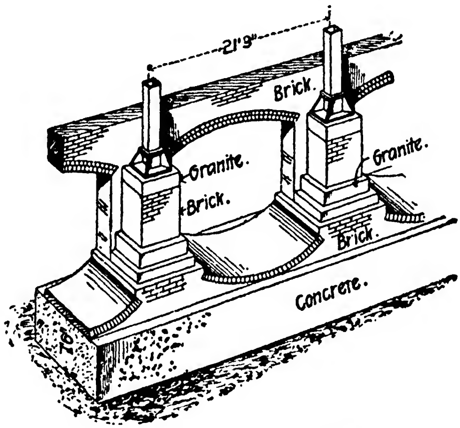


FIG. 13-1a.—Footing of Inverted Masonry Arch, Drexel Building, Philadelphia, Pa.

published in 1872. In its original and simplest form the independent footing consists of a wide concrete or masonry footing with its maximum area at the base and stepped off to decrease in horizontal area toward the top, the latter being of sufficient size to form a seat for the wall or column base.

Owing to their low bending strength, footings of masonry or of plain concrete require a very considerable depth to take care of heavy loads. Other forms have been developed that require much less depth; among these are the wooden grillage, the steel I-beam grillage, and the reinforced-concrete grillage.

Probably the first building in America to be built on a steel-grillage foundation was the Montauk Block, Chicago, built in 1881–1882 and designed by Burnham and Root, architects. The ordinary masonry footing was used for a part of the building, but, to obtain space for the boiler, a grillage of steel rails embedded in concrete was used in one part of the cellar. In Chicago the clay near the

surface has better bearing capacity than that farther down; hence it was customary to extend the footings up into the cellars, which practically filled the space with pyramids of masonry. The Tacoma Building, in the same city, completed in 1889 and taken down in 1929, was probably the first building in which I-beams, instead of steel rails, were used for the grillages.

13-2. Masonry and Timber Footings. In designing footings, the required area of base is found by dividing the load by the unit safe bearing capacity of the soil (Art. 1-15). To safeguard the masonry against crushing, the compressive unit stress on any horizontal section should not exceed the values given in Table 13-2*a*. The top of the footing is generally made a little larger than the column base or wider than the wall.

TABLE 13-2*a*

Character of Masonry	Safe Compression, Pounds per Square Inch
Concrete, 1:2:4 mixture	550
Concrete, 1:2½:5 mixture	450
Common brick, hard burned (portland cement mortar)	200
Common brick, ordinary (portland cement mortar)	175
Rubble masonry, coursed (portland cement mortar)	250
Rubble masonry, uncoursed (portland cement mortar)	200

Having determined the top and bottom areas of the footing, the next step is to design the offsets, which fix the depth of the footing. These offsets may be assumed to act as free cantilevers. Considering the case of a masonry or plain concrete footing for the wall of a building, let p denote the pressure in pounds per square foot on the base of the course in question, s the permissible bending stress in pounds per square inch, o the offset of the course in inches, and t the required thickness of the course in inches, then

$$t = o\sqrt{\frac{p}{48s}}.$$

For example, let us design a wall footing for a load of 10,000 lb. per lin. ft. of wall, the permissible soil pressure $p = 2{,}000$ lb. per sq. ft. and the permissible bending stress $s = 30$ lb. per sq. in. for 1:2½:5 concrete. The wall is 12 in. thick.

The required base width is 10,000/2,000 = 5 ft. Let us use two offset courses each 1 ft. wide to get the required 5-ft. base. Then for the lower course $t = 12\sqrt{\frac{2{,}000}{48 \times 30}} = 14.1$, say 14¼ in.;

and for the upper course $t = 12\sqrt{\dfrac{3{,}333}{48 \times 30}} = 18\frac{1}{4}$ in., or a total footing depth of 2 ft. $8\frac{1}{2}$ in.

Figure 13-2*a* illustrates an octagonal column footing of concrete, where the footing rests on piles.

A timber grillage consists of two or more layers of heavy timbers, each layer being placed at right angles to the one above and below; the top and bottom are often sheathed with a layer of planking. The several courses are well tied together with drift bolts. This type of grillage is especially suitable for use in temporary building construction, such as exposition buildings. For permanent construction the

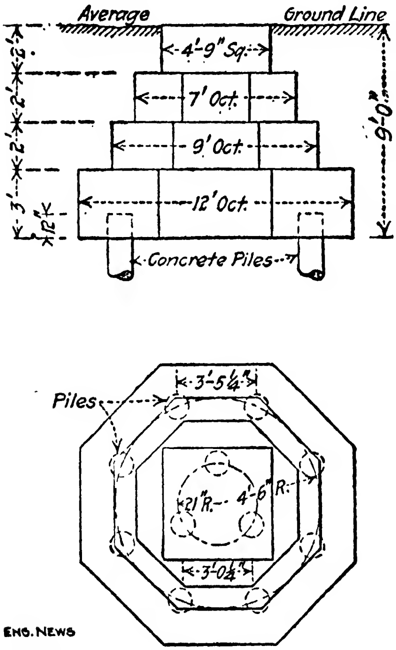


FIG. 13-2*a*.—A Typical Masonry Footing.

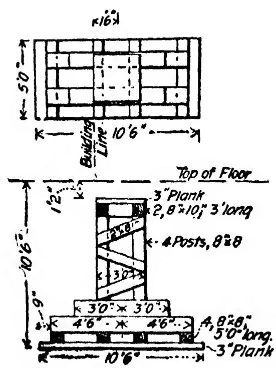


FIG. 13-2*b*.—Spread Footing of Timber Column.

wood should be creosoted, unless placed below ground-water level. Figure 13-2*b* illustrates a typical timber grillage.

A timber footing is to be designed for a column load of 19,500 lb., the column section being $9\frac{1}{2}$ in. sq. (Fig. 13-2*c*). With an allowable soil bearing pressure of 2,500 lb. per sq. ft., the required bearing area is 19,500/2,500 = 7.8 sq. ft. We will use a base 3 ft. square. Using 1,600 lb. f southern pine timber and assuming that the wood will be more or less continuously damp or wet, the unit stresses, expressed in pounds per square inch, will be taken as follows;[1] bending, 1,600 × 0.71 = 1,136; horizontal shear, 120; and bearing across the grain, 380 × 0.58 = 220.

[1] See *U. S. Dept. Agr., Mis. Pub.* 185, p. 21.

For such short beams horizontal shear will probably govern. If we use two tiers of beams and assume the lower tier to be composed of three beams $5\frac{1}{2}$ in. wide, the maximum shear in the single sill beam will be $19{,}500/3 = 6{,}500$ lb. Denoting the width of the beam as b and the depth as d, then $2bd/3 = 6{,}500/120$, or $bd = 81.2$ sq. in. Hence, a 10- by 10-in. beam (dressed to $9\frac{1}{2}$ by $9\frac{1}{2}$) will be used, with $bd = 90.25$ sq. in. The maximum moment is at the center, where $M = 6{,}500 \times 15.25 + 3{,}250 \times 1.37 - 9{,}750 \times 2.37 = 80{,}500$ in.-lb. Hence the maximum bending stress is $6 \times 80{,}500/\overline{9.5}^3 = 564$ lb. per sq. in. The bearing of the column on the sill beam is $19{,}500/\overline{9.5}^2 = 216$ lb. per sq. in.

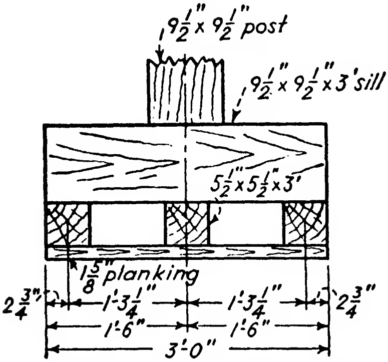


FIG. 13-2c.—Timber Beam Grillage Footing.

For the lower tier of beams the maximum shear is $6{,}500 \times 13.25/36 = 2{,}390$ lb.; hence $2bd/3 = 2{,}390/120$, or $bd = 29.9$ sq. in. A 6- by 6-in. section (dressed to $5\frac{1}{2}$ by $5\frac{1}{2}$) furnishes 30.25 sq. in. The maximum moment is $3{,}250(9 - 2.37) = 21{,}550$ in.-lb. and the maximum bending stress $6 \times 21{,}550/5.5^3 = 777$ lb. per sq. in. The bearing of the sill beam on the beams below is $6{,}500/(9.5 \times 5.5) = 124$ lb. per sq. in.

The lower tier of beams will be sheathed on the bottom with 2-in. planking dressed to $1\frac{5}{8}$ in.

13-3. Designing Loads. Loads coming to basement columns and footings are dead, live, and wind. Dead loads, which include the weight of the frame and floors of the building, can be closely figured. Live and wind are assumed loads, which in most cities are governed by building codes. The New York City Code (1938) for office buildings specifies a minimum live footing load, in pounds per square foot, of 40 for the roof where the rise is 3 in. or less per foot and 40 for the top floor, the load on each succeeding floor being reduced by 2.5 until the figure reaches 25, after which there is no further reduction.

In spread foundations a moderate settlement is to be expected, but care must be exercised to make this uniform. However, this objective is difficult of attainment where the percentage of live to total load differs considerably for the several footings. Dead load usually causes the greater part of the total settlement, as it is the

first load to be applied and remains at its full value. The actual live load is usually much less than the design figure, and even this comes on only at intervals. For these reasons it is customary to design footings for a uniform unit pressure under dead load or under dead plus *partial* live. Many building codes omit wind in the design of footings, and except for high and narrow buildings this would seem to be good practice.

A formula used for footing design is

$$A = \frac{KT}{K_1 B}$$

where A denotes the required area of any footing, B the allowable unit pressure on the soil, T the total load on that footing having the largest ratio of live to dead load, K the dead plus one-third live load on the footing for which the area is being found, and K_1 the dead plus one-third live load on that footing having the largest ratio of live to dead load. All dimensions are in feet and all forces in pounds. The use of this formula will give equal intensities of pressure on all footings under dead plus one-third live loads, and no footing will have a pressure under dead plus full live load greater than the allowable unit pressure.

The following table gives the results of applying this formula to four columns of an actual structure where the allowable soil pressure was 7,000 lb. per sq. ft. It will be observed that under dead plus one-third live load the unit soil pressure is the same for all columns and equals 5,130 lb. per sq. ft. For dead plus full live load the values range from 5,410 to 7,000, and for dead load they range from 4,190 to 4,980.

	(44)	(1)	(29)	(24)
Dead load, pounds	435,000	375,000	429,000	379,000
Live load, pounds	38,000	230,000	159,000	253,000
Dead + $\frac{1}{3}$ live load, pounds	447,700	457,700	482,000	463,300
Required area, square feet	87.4	89.4	94.0	90.3
Unit pressure, dead + $\frac{1}{3}$ live	5,130	5,130	5,130	5,130
Unit pressure, dead + live	5,410	6,770	6,260	7,000
Unit pressure, dead	4,980	4,190	4,570	4,200

For a complete discussion on this subject the reader is referred to *Engineering News*, vol. 69, p. 463, Mar. 6, 1913; and to *Engineering News-Record*, vol. 85, p. 219, July 29, 1920.

13-4. Design of I-beam Grillages. In the construction of I-beam grillages two or more tiers are used, the exact number depending on the desired spread of base. Each tier is placed at right angles to the one below it and the load is carried to the soil through beam action. The individual beams of each tier should be held in place by cast-iron or gas-pipe separators, preferably the former. These separators should be placed near each end of the beams and at intermediate positions not over 5 ft. apart. The beams should be spaced so as to give a clearance of not less than 3 in., in order that concrete may readily be filled in between the beams. Preferably the clearance should not materially exceed one and one-half times the width of the flange.

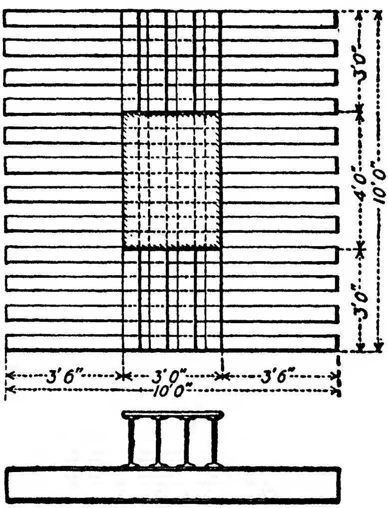


FIG. 13-4*a*.—Steel I-beam Grillage for a Single Column.

Concrete should be filled in between the beams and also placed around the sides, top, and bottom of the grillage. The thickness of the bottom layer should not be less than 12 in., and the top and sides should have a protective coating of at least 4 in. net thickness.

In designing steel grillage foundations, the following assumptions are made: (*a*) the pressure from the footing is uniformly distributed over the bed, (*b*) the pressure of one tier of beams is uniformly distributed on the tier below, (*c*) each tier acts independently of all other tiers, and (*d*) the concrete filling and covering carry no stress, acting merely as a protection against corrosion.

For the single-column grillage the square base is the most economical shape. Where the possible width is restricted, as in wall-column footings, the grillage should be made as nearly square as possible. Economy also results in using a minimum number of tiers.

EXAMPLE OF DESIGN OF SINGLE-COLUMN FOOTING (FIG. 13-4*a*). Load = 600,000 lb. Allowable pressure on foundation bed = 6,000 lb. per sq. ft. Size of column base = 3 by 4 ft. Required area of base = 600,000/6,000 = 100 sq. ft. A base 10 ft. square is adopted. Assume two tiers of beams. For the top tier, the maximum bending moment $M = \frac{600{,}000}{2} \times \frac{5-2}{2} \times 12$

= 5,400,000 lb.-in. Using 18,000 lb. per sq. in. as the safe unit stress in the outer fiber, the total section modulus required = S = 5,400,000/18,000 = 300 in.^{3} Trying various combinations of beams, the following results are obtained:

No.	Number of beams	S required	Size of beam Inches	Size of beam Pounds	S furnished	Width of flange, inches	Clearance, inches
1	3	100	20	65.4	116.9	6.25	8.6
2	4	75	18	54.7	88.4	6.00	4.0
3	5	60	15	45	60.5	5.54	2.1

The choice lies between Nos. 1 and 2, since No. 3 does not give sufficient clearance. The weight favors No. 1, being 226 lb. lighter, but No. 2 will be used as it gives a more satisfactory clearance and has less depth, thus saving on concrete filling and also excavation.

For the lower tier: Maximum $M = \dfrac{600{,}000}{2} \times \dfrac{5 - 1.5}{2} \times 12 = 6{,}300{,}000$ lb.-in. Total required S = 6,300,000/18,000 = 350 in.^{3} The following results are obtained by trying various combinations of beams:

No.	Number of beams	S required	Size of beam Inches	Size of beam Pounds	S furnished	Width of flange, inches	Clearance, inches
1	10	35	12	31.8	36.0	5.00	7.8
2	12	29.2	12	31.8	36.0	5.00	5.5
3	14	25	12	31.8	36.0	5.00	3.8
4	16	21.9	10	25.4	24.4	4.66	3.0

Number 2 will be used. After designing for bending, the beams should be checked for shearing and web buckling due to concentration of applied loads. The maximum shear in the upper tier is $600{,}000 \times \frac{3}{10} = 180{,}000$ lb. The safe shearing strength, based on a unit value of 12,000 lb. per sq. in. and uniform distribution over the web, is $4 \times 12{,}000 \times 18 \times 0.46 = 398{,}000$ lb. The maximum shear on the lower tier is $600{,}000 \times \dfrac{3.5}{10} = 210{,}000$ lb., while the shearing capacity is $12 \times 12{,}000 \times 12 \times 0.35 = 605{,}000$ lb.

A vertical load may be considered to be distributed over a horizontal length of the web equal to the length over which the superimposed load is distributed plus one-half the depth of the beam. On this basis the unit compression for the upper tier of beams is $\dfrac{600{,}000}{(48 + 9) \times 4 \times 0.46} = 5{,}730$ lb. per sq. in. The allowable unit stress from the commonly used column formula $f = 15{,}000 - \dfrac{1}{4}\left(\dfrac{l}{r}\right)^2$ is $f = 15{,}000 - \dfrac{1}{4}\left(\dfrac{18}{0.29 \times 0.46}\right)^2 = 10{,}500$ lb. per sq. in. Likewise, for the lower tier the unit buckling stress is $\dfrac{600{,}000}{(36 + 6) \times 12 \times 0.35} = 3{,}400$

lb., while the safe unit stress is $15{,}000 - \frac{1}{4}\left(\frac{12}{0.29 \times 0.35}\right)^2 = 11{,}500$ lb. per sq. in.

Since in grillage work the maximum moment and maximum shear occur at sections not far apart, in close designing it may sometimes be necessary to investigate the combined stresses, both tension and shear, at the fillet of the beam on the section of maximum shear. The maximum bending stress in tension is

$$t_{\max} = \frac{t}{2} + \sqrt{s^2 + \frac{t^2}{4}}$$

and the maximum unit shear is

$$s_{\max} = \sqrt{s^2 + \frac{t^2}{4}}$$

where t = intensity of bending tension
s = intensity of vertical shear

13-5. Design of Two- and Three-column Footings. Where the two-column loads are equal, the base of the footing should be rectangular in shape and symmetrical about a line midway between the

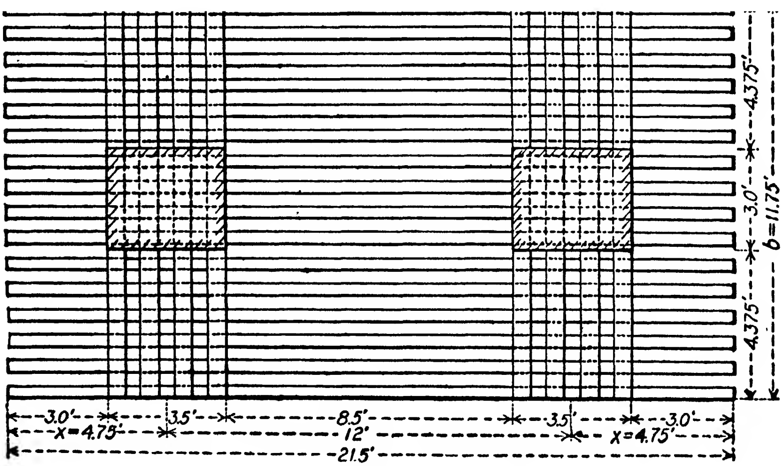


FIG. 13-5*a*.—Double-column Footing of Steel I-beams.

columns. When the total area of the base has been determined and the distance between columns fixed, the proportion of length to breadth for the base of footing should be such that the moment in the lower tier of beams under the column centers equals that at a point midway between the columns. This makes the three maximum moments approximately equal and gives the greatest economy of material.

EXAMPLE OF DESIGN OF DOUBLE-COLUMN FOOTING, EQUAL LOADS (FIG. 13-5a). Column loads each 500,000 lb. Column spacing = 12 ft. Allowable pressure on ground = 4,000 lb. per sq. ft. Size of column bases = $3\frac{1}{2}$ by 3 ft. Allowable unit stress in beams = 18,000 lb. per sq. in. To get the value x that will make the three moments equal, $500{,}000(6 - x)/2 = 500{,}000x^2/2(6 + x) - 500{,}000(\frac{7}{16})$, whence $x = 4.77$ ft. Required bearing area of base = 1,000,000/4,000 = 250 sq. ft. Using a value of x of 4.75 ft., $b = 250/(12 + 2 \times 4.75) = 11.63$ ft.; say 11.75. Let two tiers of beams be assumed. Computing for top tier: Maximum $M = 500{,}000(11.75 - 3)\frac{12}{8} = 6{,}560{,}000$ lb.-in. Total required $S = 6{,}560{,}000/18{,}000 = 364$ in.^{3} After trying various combinations of beams, the results are as follows:

No.	Number of beams	S required	Size of beam Inches	Size of beam Pounds	S furnished	Width of flange, inches	Clearance, inches
1	3	121.3	20	70	121.4	6.32	11.5
2	4	91	18	60	93.1	6.09	5.9
3	5	72.8	18	54.7	88.4	6.0	3.0

No. 2 will be adopted.

For the lower tier the three positions of maximum bending moment are at the center and 4.45 ft. from each end. M at center = $500{,}000(6 - 5.375)12 = 3{,}750{,}000$ lb.-in. M at 4.45 ft. from the end =

$$\left(\frac{500{,}000}{10.75} \cdot \frac{4.45^2}{2} - \frac{500{,}000}{3.5} \cdot \frac{\overline{1.45}^2}{2}\right) 12 = 3{,}720{,}000 \text{ lb.-in.}$$

Total required $S = 3{,}750{,}000/18{,}000 = 208$. Upon trying various combinations of beams, the results are found to be as follows:

No.	Number of beams	S required	Size of beam Inches	Size of beam Pounds	S furnished	Width of flange, inches	Clearance, inches
1	12	17.3	10	25.4	24.4	4.66	7.7
2	14	14 8	8	20.5	15.1	4.08	6.5
3	16	13	8	18.4	14.2	4.0	5.1
4	18	11.5	8	18.4	14.2	4.0	4.1

No. 3 will be adopted.

When the column loads are not equal, the center of gravity of the base of the grillage is usually made to coincide with the line of action of the resultant of the two column loads by making the base a trapezoid; or, if the loads are nearly equal, it may be done by using a rectangular shape and making the cantilever end at the heavy load longer than the other cantilever end. The trapezoidal shape may be obtained either by using a larger number of beams at the heavy load end or by using the same number of beams and

spacing them more closely at one end than at the other. A combination of the two methods is sometimes used.

If the proportions of the base are so fixed that the bending moment under the center of each column equals that at the center of gravity of the base, the three maximum moments in the lower tier of grillage will be closely equal; this condition gives approximately the minimum amount of material. The most satisfactory method of determining the dimensions to secure this result is by trial.

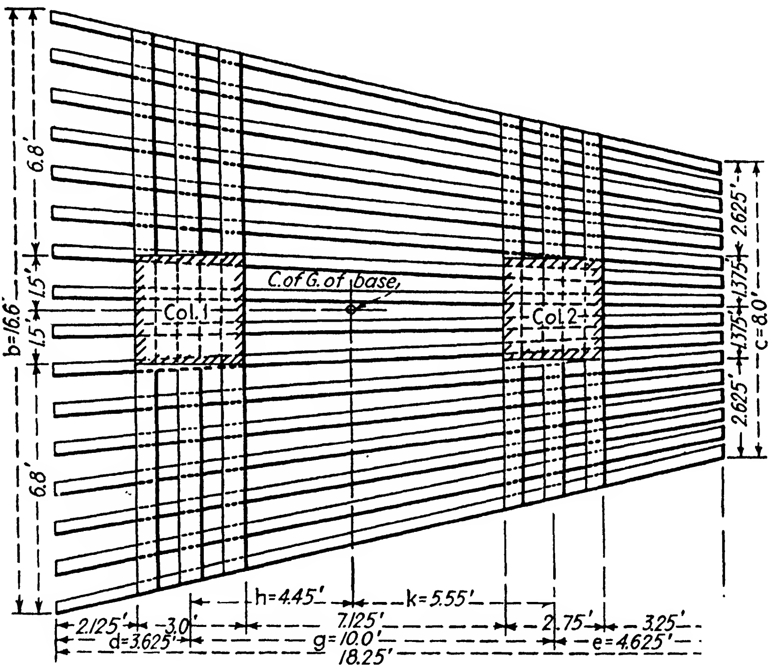


FIG. 13-5*b*.—Steel I-beam Grillage for Two Columns Supporting Unequal Loads. The Load on Column 1 is 500,000 Lb.; That on Column 2 is 400,000 Lb.

EXAMPLE OF DESIGN OF DOUBLE-COLUMN FOOTING, UNEQUAL LOADS. Column loads and spacing as shown in Fig. 13-5*b*. Allowable pressure on foundation bed = 4,000 lb. per sq. ft. Size of column bases as shown in Fig. 13-5*b*. Allowable unit stress in beams = 18,000 lb. per sq. in. Required bearing area of base = 900,000/4,000 = 225 sq. ft. Distance from column 1 to resultant of both column loads = 400,000 × 10/900,000 = 4.45 ft.

Assuming d = 3.625 and e = 4.625, the values of b and c are given by the formula $\frac{A}{L}\left(1 \pm 6\frac{f}{L}\right)$, the plus sign for b and the minus for c, where A denotes the area of the base, L the length of the base, and f the distance from the center of the base to the center of gravity of the loads.

Substituting in this formula,

$$b = \frac{225}{18.25}\left(1 + \frac{6 \times 1.05}{18.25}\right) = 16.6 \text{ ft.}$$

and

$$c = \frac{225}{18.25}\left(1 - \frac{6 \times 1.05}{18.25}\right) = 8.0 \text{ ft.}$$

Letting y_1 denote the breadth of the footing at the center of column 1, $y_1 = c + (b - c)(e + g)/L = 8 + 8.6 \times 14.62/18.25 = 14.88$ ft.

The moment under the center of column 1 is $4{,}000(2b + y_1)d^2/6 - 500{,}000 \times \frac{3}{8} = 4{,}000(2 \times 16.6 + 14.88)\ 3.625^2/6 - 187{,}500 = 233{,}700$ ft.-lb.

Letting y_3 denote the breadth of footing at center of gravity $y_3 = c + (b - c)(e + k)/L = 8 + 8.6 \times 10.17/18.25 = 12.8$ ft.

The moment at the center of gravity of the loads is

$$\frac{4{,}000(2b + y_3)(d + h)^2}{6} - 500{,}000 \times 4.45$$

$$= \frac{4{,}000(2 \times 16.6 + 12.8)\overline{8.07}^2}{6} - 2{,}225{,}000 = -225{,}000 \text{ ft.-lb.}$$

Letting y_2 denote the breadth of the footing at the center of column 2,

$$y_2 = c + \frac{(b - c)e}{L} = 8 + 8.6 \times \frac{4.62}{18.25} = 10.18 \text{ ft.}$$

The moment under the center of column 2 is $4{,}000(y_2 + 2c)e^2/6 - 400{,}000 \times 2.75/8 = 4{,}000(10.18 + 2 \times 8)\overline{4.62}^2/6 - 137{,}500 = 236{,}000$ ft.-lb.

These moments, being nearly equal, show that d and e were correctly assumed.

Using two tiers of beams, the computations for the upper tier under column 1 give the following:

Maximum $M = (500{,}000/8)(14.88 - 3)12 = 8{,}910{,}000$ lb.-in. Total required $S = 495$ in.[3] After trying various combinations of beams, the results are as follows, and No. 1 is adopted:

No.	Number of beams	S required	Size of beam Inches	Pounds	S furnished	Width of flange, inches	Clearance, inches
1	3	165	24	79.9	173.9	7.00	7.5
2	4	123.8	20	75	126.3	6.39	3.4

In designing the lower tier, letting x = the distance from the left end of grillage to the section in question, the expression for the moment of the upward forces is $\frac{4{,}000x^2}{6}\left[3b - \frac{(b - c)x}{L}\right]$. Hence, the expressions for bending moments under column 1, between the two columns and under column 2, are respectively as follows:

$$M \text{ (column 1)} = \frac{4{,}000}{2} \cdot \frac{x^2}{3} \cdot (49.8 - 0.471x) - \frac{500{,}000}{3} \cdot \frac{(x - 2.125)^2}{2}.$$

$$M \text{ (between columns)} = \frac{4{,}000}{2} \cdot \frac{x^2}{3} \cdot (49.8 - 0.471x) - 500{,}000(x - 3.625).$$

$$M \text{ (column 2)} = \frac{4{,}000}{2} \cdot \frac{x^2}{3} \cdot (49.8 - 0.471x)$$

$$- 500{,}000(x - 3.625) - \frac{400{,}000}{2.75} \cdot \frac{(x - 12.25)^2}{2}.$$

To get the values of x for the maximum value of M in each of the above equations, equate dM/dx to zero, which gives 3.42, 8.58, and 13.91 ft., respectively. Substituting these three values of x in the preceding equations, the corresponding values of M are 236,000, 231,000, and 236,000 ft.-lb. The maximum maximorum is, therefore, 236,000 ft.-lb. Total required S = 157.5 in.^{3} Trying various combinations of beams, there is obtained the following:

No.	Number of beams	S required	Size of beam Inches	Pounds	S furnished	Width of flange, inches	Clearance, inches
1	10	15.8	8	23	16	4.17	6–17.5
2	12	13.2	8	18.4	14.2	4.00	4.4–13.7
3	14	11.3	8	18.4	14.2	4.00	3.1–11.0
4	16	9.9	7	15.3	10.4	3.66	2.5– 9.4

No. 4 will be adopted.

Analysis will show that both foregoing designs are safe in shear and buckling.

Reinforcing bars should be placed in the concrete near the upper surface for the wider half of the footing.

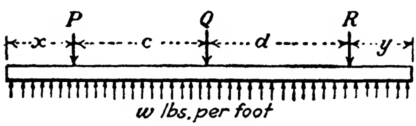


Fig. 13-5c.

Where more than two columns have a common footing, the structure becomes statically indeterminate and unsolvable unless some assumption is made regarding the deflection of the beams at the column bases. On the assumption that the points of application of P, Q, and R of Fig. 13-5c remain in a straight line after deformation of the grillage, it can be proved by deflection formulas that

$$\frac{8c^3P - w(6x^2c^2 + 8xc^3 + 3c^4)}{8d^3R - w(6y^2d^2 + 8yd^3 + 3d^4)} = -\frac{c}{d}.$$

Applying the two laws $\Sigma Y = 0$ and $\Sigma M = 0$ to the structure taken as a free body, together with this equation, the values of x, y, and w can be found.

As an example, let the loads P, Q, and R be, respectively, 600, 900, and 1,500 tons, $c = 12$ ft. and $d = 30$ ft. The values of x, y, and w are found to be 7.8 ft., 15 ft., and 46.3 tons, respectively. If the safe bearing capacity of the soil is 5 tons per sq. ft., the width of footing would be 9.5 ft.

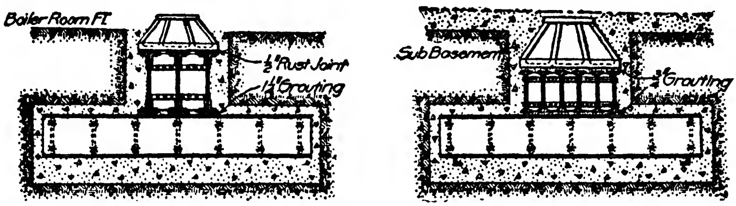


FIG. 13-6a.—Footings with Plate Girders and I-beams in Double Tiers.

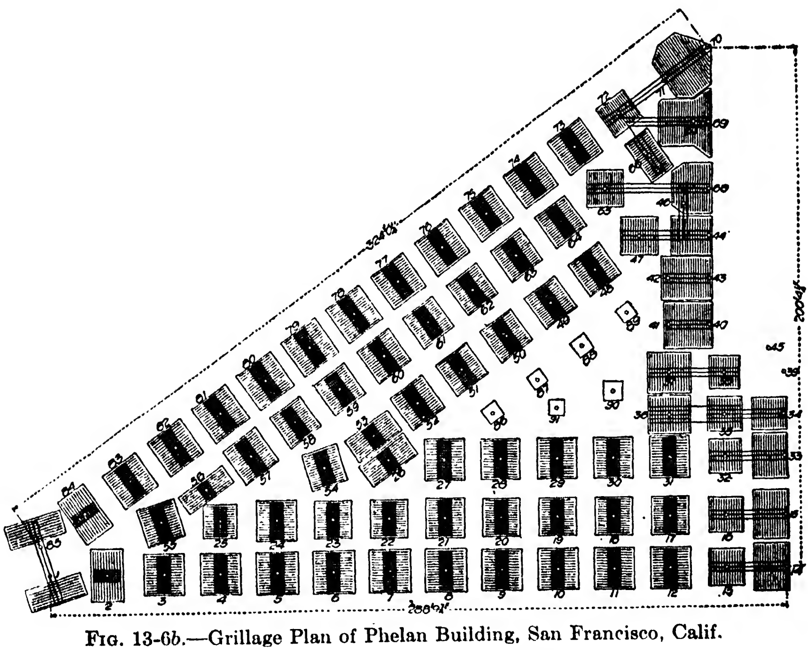


FIG. 13-6b.—Grillage Plan of Phelan Building, San Francisco, Calif.

13-6. Examples of Steel-grillage Foundations. Most of the grillages used in the foundations of the Phelan Building, San Francisco, were 15 ft. square and made with two cross tiers of I-beams from 18 to 24 in. in depth as shown in the right-hand drawing of Fig. 13-6a, or with an upper tier of built-up girders and a lower

tier of I-beams as shown in the left-hand drawing of the same figure. The complete grillage plan is shown in Fig. 13-6*b*.

The lower tier of beams were set in exact position by first pouring a bed of concrete 12 in. thick. On the top of this concrete there were two full-length grooves transverse to the beams. In each groove a 3- by 3- by $\frac{5}{16}$-in. angle was carefully leveled with the

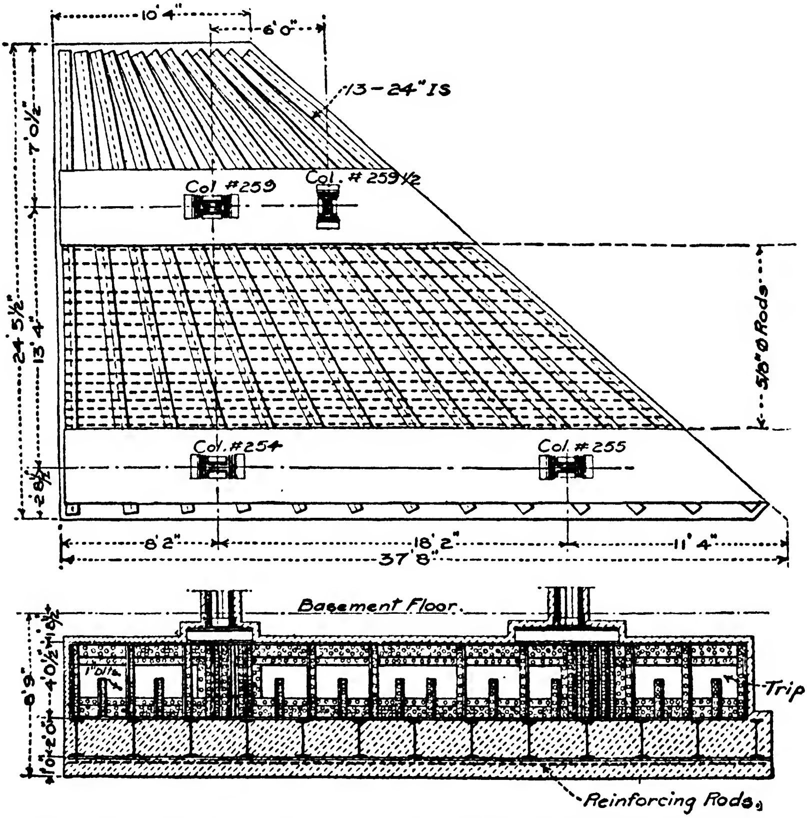


FIG. 13-6c.—Special Footing for Four Columns, Curtis Building, Philadelphia.

upper edge of its vertical flange horizontal and $\frac{3}{4}$ in. above the surface of the concrete. These served as leveling bars to receive the lower flanges of the grillage beams and to ensure their exact height. The spaces between the beams and the concrete were grouted, after which the upper tier was placed, being shimmed $\frac{3}{4}$ in. above the top flanges of the lower tier, and grouted. The pedestals were placed in the same manner, and a solid mass of concrete was placed, affording a 6-in. thickness of insulation all around the metal. At the

present time it is quite general practice to use steel slabs rather than metal pedestals to distribute the load from the column to the grillage.

Figure 13-6c illustrates the very heavy grillage foundation used for four columns of the Curtis Building, Philadelphia. It was necessary to use a single grillage because of the short distance between the columns. The distributing girders for columns 254 and 255 have 48-

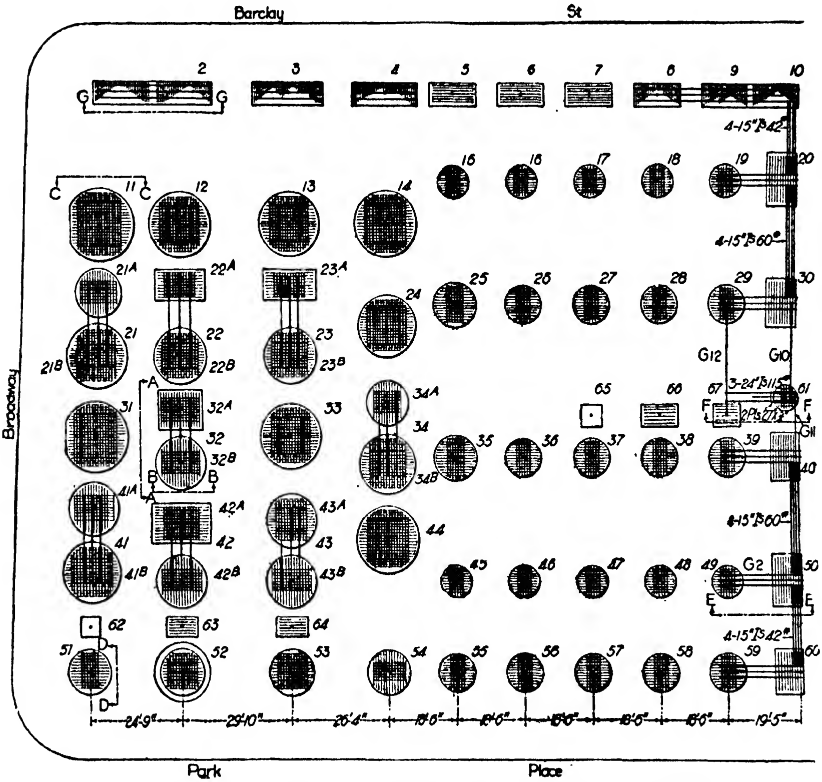


FIG. 13-6d.—Plan of Piers and Grillages for the Woolworth Building.

by $\frac{11}{16}$-in. webs reinforced by 5- by 3- by $\frac{3}{8}$-in. vertical stiffener angles and two 13- by $\frac{1}{2}$-in. vertical side plates, and the top flanges of the girders are connected by transverse tie plates. The column loads are transmitted to the triple distributing girders by bolsters made of solid slabs of plain square steel billets which are bolted to the upper flanges of the girders. The concrete footing is reinforced with rods for part of the base, due to the fact that the I-beams are a considerable distance apart, thus developing beam action in the concrete.

The Woolworth Building, New York City, is founded on solid rock 115 ft. below curb level. The loads are carried from the columns to bedrock through grillage footings resting on reinforced-

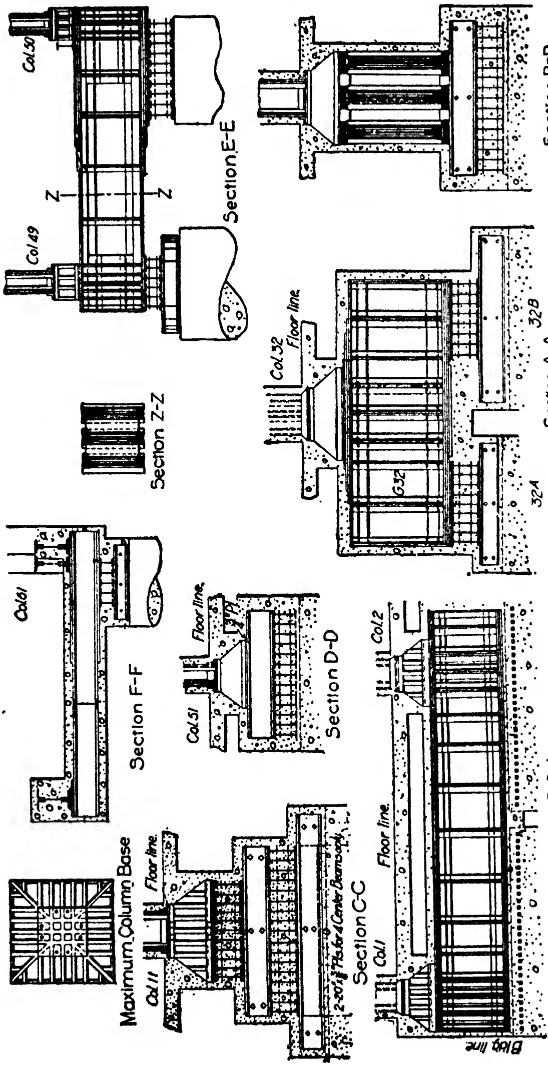


Fig. 13-6*e*.—Column Footings of Plate Girders and Grillages, Woolworth Building, New York City. For column 11 the diameter of the pier is 18 ft. 9 in. and the column load is 9,423,000 lb. For this column the grillage is made for four tiers of 24-in. I-beams with flange reinforcement plates for the top tier and web reinforcement plates for the bottom tier. Section *E-E* shows the common method of supporting wall columns when the foundation must be kept back of the property line.

concrete piers. Figure 13-6*d* shows the general layout for the foundation, while Fig. 13-6*e* illustrates some of the details. For column 11 the diameter of the pier is 18 ft. 9 in., and the column load is 9,423,000 lb. For this column the grillage is made of four tiers of 24-in. I-beams with flange reinforcement plates for the top

tier and web reinforcement plates for the bottom tier. Section *E-E* shows the common method of supporting wall columns when the foundation must be kept back of the property line.

13-7. Design of Reinforced-concrete Wall Footings. Instead of serving merely as a protection for the steel, concrete may be made to take the load by using a reinforced-concrete footing in place of the I-beam grillage, thus perhaps lessening the cost of the foundation. Another advantage possessed by a reinforced-concrete foundation is that it can be cast in any shape or form desired.

As an example of wall-footing design, assuming the load to be 80,000 lb. per lin. ft. of wall and the allowable bearing on the soil 5,000 lb. per sq. ft., the width of footing will be 80,000/5,000 = 16

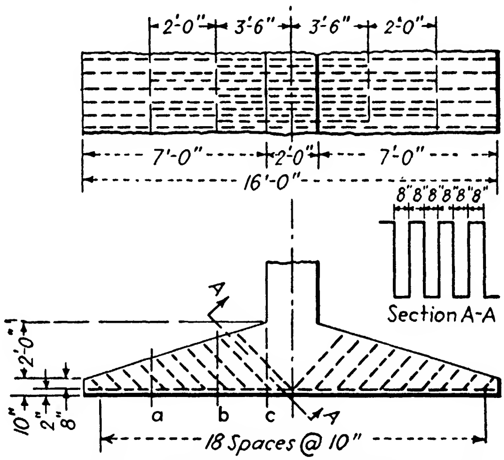


FIG. 13-7*a*.—Reinforced-concrete Wall Footing.

ft. The thickness of the wall is 2 ft. (Fig. 13-7*a*). The footing will be designed at three sections: at *a*, $5\frac{1}{2}$ ft. from the center of the wall; at *b*, 3 ft. from the center; and at *c*, 1 ft. from the center. Taking a 1-ft. length of footing, the vertical shears and bending moments will be as follows:

$$V_a = 5{,}000 \times 2\frac{1}{2} = 12{,}500 \text{ lb.} \qquad M_a = \frac{5{,}000 \times \overline{2.5}^2 \times 12}{2} = 187{,}500 \text{ lb.-in.}$$

$$V_b = 5{,}000 \times 5 = 25{,}000 \text{ lb.} \qquad M_b = \frac{5{,}000 \times \overline{5}^2 \times 12}{2} = 750{,}000 \text{ lb.-in.}$$

$$V_c = 5{,}000 \times 7 = 35{,}000 \text{ lb.} \qquad M_c = \frac{5{,}000 \times \overline{7}^2 \times 12}{2} = 1{,}470{,}000 \text{ lb.-in.}$$

Let the permissible stresses be as follows, all values being in pounds per square inch: compression in extreme fiber of concrete on plane carrying maximum intensity, $f_{c_{max}} = 875$; diagonal tension in concrete, $t = 50$; tension in steel reinforcement, $f_s = 18{,}000$; and bond in steel reinforcement, $u = 150$. The ratio of the modulus of elasticity of steel to that of concrete will be assumed as $n = 12$. Using standard nomenclature of reinforced-concrete design, we have for the required depth of beam to center of steel, $d = \sqrt{M/Rb}$, in which $R = f_c kj/2$. In the latter formula $k = \dfrac{1}{1 + f_s/nf_c}$ and $j = 1 - k/3$. We also have $p = \dfrac{1}{2}\dfrac{1}{\dfrac{f_s}{f_c}\left(\dfrac{f_s}{nf_c} + 1\right)}$. The work involved in getting the value of R will be greatly reduced by using diagrams found in textbooks[1] on concrete design.

Assuming $f_c = 800$ lb. per sq. in., the value[2] of R is found to be 123. Solving for d, $d_a = \sqrt{187{,}500/(123 \times 12)} = 11.3$ in.; $d_b = \sqrt{750{,}000/(123 \times 12)} = 22.6$ in.; and

$$d_c = \sqrt{\frac{1{,}470{,}000}{(123 \times 12)}} = 31.5 \text{ in.}$$

As it is inadvisable to use a depth at any section less than about 8 in., the form shown in Fig. 13-7*a* will be adopted. Here $\tan \alpha = \frac{24}{84} = 0.286$ and $\sec \alpha = \sqrt{1 + \tan^2 \alpha} = \sqrt{1 + 0.286^2} = 1.04$; therefore, $f_c = 875/1.04^2 = 810$ lb. per sq. in.; hence our assumed value is satisfactory.

The area of steel required at each section is given by the formula $A = pbd$, where $p = \dfrac{1}{2}\dfrac{1}{\dfrac{18{,}000}{800}\left(\dfrac{18{,}000}{12 \times 800} + 1\right)} = 0.00774$. Hence

$$A_a = 0.00774 \times 12 \times 11.3 = 1.05 \text{ sq. in.}$$
$$A_b = 0.00774 \times 12 \times 22.6 = 2.10 \text{ sq. in.}$$
$$A_c = 0.00774 \times 12 \times 31.5 = 2.92 \text{ sq. in.}$$

Using a rod spacing of 3 in. center to center, the required area of each rod at section *c* will be $2.92/4 = 0.73$ sq. in. A 1-in. round

[1] For example, "Reinforced-concrete Construction," by Turneaure and Maurer.

[2] In a wedge-shaped beam the greater principal stress at the outer fiber acts parallel to the upper surface of the beam. The stress on a vertical plane at this point has an intensity of $f_c = f_{c_{max}} \cos^2 \alpha$, in which α is the angle of inclination of the upper surface of the beam.

deformed rod, giving an area of 0.785 sq. in., will be adopted. Three rods per foot of wall will furnish more than the required area at b, and two rods will furnish more than the required area at a; hence the rods will be arranged as shown in the upper drawing of Fig. 13-7a.

In a wedge-shaped beam the formula for unit bond stress[1] is $u = (Vd - M \tan \alpha)/(j\, d^2 \Sigma O)$; hence, using for d the actual depth rather than the required depth,

$$u_a = \frac{12{,}500 \times 16.6 - 187{,}500 \times 0.286}{0.88 \times \overline{16.6}^2 \times 6.28} = 101 \text{ lb. per sq. in.}$$

$$u_b = \frac{25{,}000 \times 25.2 - 750{,}000 \times 0.286}{0.88 \times \overline{25.2}^2 \times 12.56} = 59 \text{ lb. per sq. in.}$$

$$u_c = \frac{35{,}000 \times 32 - 1{,}470{,}000 \times 0.286}{0.88 \times \overline{32}^2 \times 12.56} = 62 \text{ lb. per sq. in.}$$

All these values are below the safe limit of 150 lb. per sq. in.

The intensity of diagonal tension t may be found by multiplying the several bond stress values by the ratio of the perimeters of the rods to the breadth of the beam; thus

$$t_a = 101 \times \frac{6.28}{12} = 53 \text{ lb. per sq. in.}$$

$$t_b = 59 \times \frac{12.56}{12} = 62 \text{ lb. per sq. in.}$$

$$t_c = 62 \times \frac{12.56}{12} = 65 \text{ lb. per sq. in.}$$

A stirrup system will be designed to take the diagonal tension in excess of 50 lb. per sq. in., the stirrups being placed on a 45-deg. slope. If we use $\frac{1}{4}$-in. round deformed rods with prongs spaced 8 in. as shown in Fig. 13-7a, the strength of one line of stirrups in a 12-in. length will be $18{,}000 \times 0.0491 \times \frac{12}{8} = 1{,}326$ lb. Denoting the horizontal distance between stirrup units by s, the formula is $s = 1{,}326/(12 \times t \times \cos 45 \text{ deg.})$, hence

$$s_a = \frac{1{,}326}{12 \times 3 \times 0.707} = 52 \text{ in.}$$

$$s_b = \frac{1{,}326}{12 \times 12 \times 0.707} = 13 \text{ in.}$$

$$s_c = \frac{1{,}326}{12 \times 15 \times 0.707} = 10.4 \text{ in.}$$

A uniform spacing of 10 in. will be adopted.

[1] See *Eng. Contracting*, vol. 50, p. 593, Dec. 25, 1918.

In a wedge-shaped beam the maximum intensity of vertical and horizontal shears occurs at the top and equals f_c tan α, or, for this problem $810 \times 0.286 = 234$ lb. per sq. in. In beams of constant cross section the allowable value is usually limited to about 125 lb. per sq. in., in order to keep the unit diagonal tension down to a safe figure. However, in our case, at the point of maximum shear there is no diagonal tension, and, since the unit shearing strength of concrete is approximately one-half its compression strength, any design is safe in which tan α is less than 0.5.

13-8. Design of Reinforced-concrete Column Footings. Stresses in column footings are statically indeterminate, for such footings act essentially as flat slabs. As a consequence empirical rules are used for proportioning the concrete and the reinforcement, these rules being based on a cantilever beam action. The maximum bending moment and

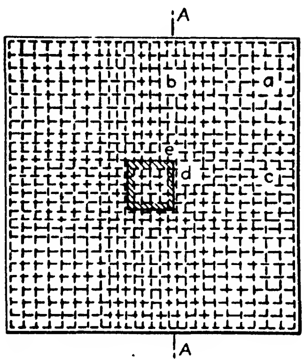


FIG. 13-8a.—Reinforced-concrete Column Footing.

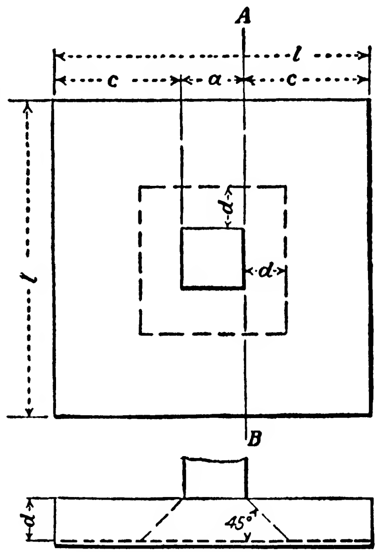


FIG. 13-8b.

shear for bond stress are generally assumed to be on a vertical plane passing through the face of the column, such as the plane AA of Fig. 13-8a, and to include all the forces outside of the section.

With two-way reinforcement it is evident that the stresses cannot be uniformly distributed along the length of this section. For example, the load from the soil at c will go to the column by dc acting as a cantilever beam. On the other hand, if we consider the upward soil pressure at a, a part will go to some point such as c through beam action along dc; while another part will go to some such point as b through ab acting as a beam. That part which goes to c will then go to the column by beam action of cd, and that part

at a will go by beam action along bc. Thus it is evident that the stresses along the plane AA will vary from a maximum at the column to a minimum near the sides of the footing.

The section resisting bending may be taken as the full vertical section except in sloped footings where the slope exceeds 1 vertical to 2 horizontal. The tensile reinforcement may be determined on the basis of 85 per cent of the moment computed.

The critical section for diagonal tension is usually taken as the vertical section formed on the square which lies at a distance from the face of the column equal to the depth of the footing. This section is shown by the dotted line of the plan figure of Fig. 13-8b.

If we refer to Fig. 13-8b and let w denote the upward unit soil pressure, all distances being in feet,

$$M = \tfrac{1}{2}wlc^2,$$
$$V \text{ for bond} = wlc.$$
$$V \text{ for diagonal tension} = w[l^2 - (a + 2d)^2].$$

Let it be required to design a two-way reinforcement footing for a 30- by 30-in. column on which the load is 432,000 lb., the allowable bearing on the soil being 3,000 lb. per sq. ft. The required footing area is 432,000/3,000 = 144 sq. ft. We shall use a 12- by 12-ft. base; then l = 12 ft., a = 2.5 ft., c = 4.75 ft., and, if we assume d = 1.333 ft.,

$$M = \tfrac{1}{2} \times 3{,}000 \times 12 \times \overline{4.75}^2 = 406{,}000 \text{ ft.-lb.}$$
$$V \text{ for bond} = 3{,}000 \times 12 \times 4.75 = 171{,}000 \text{ lb.}$$
$$V \text{ for diagonal tension} = 3{,}000[\overline{12}^2 - (2.5 + 2 \times 1.33)^2] = 352{,}000 \text{ lb.}$$

If we use the same unit stresses as in Art. 13-7 and a slab of constant thickness,

$$p = \frac{1}{2}\frac{1}{\dfrac{18{,}000}{875}\left[\dfrac{18{,}000}{875 \times 12} + 1\right]} = 0.0090.$$

$$k = \frac{1}{1 + \dfrac{18{,}000}{12 \times 875}} = 0.37.$$

$$j = 1 - \frac{0.37}{3} = 0.877.$$

$$R = \tfrac{1}{2} \times 875 \times 0.37 \times 0.877 = 142.$$

$$d = \sqrt{\frac{406{,}200 \times 12}{142 \times 12 \times 12}} = 15.4 \text{ in., say 16.}$$

$$A = 0.0090 \times 12 \times 12 \times 15.4 \times 0.85 = 17.0 \text{ sq. in.}$$

Thirty $\frac{7}{8}$-in. round bars spaced at $4\frac{3}{4}$ in. will furnish an area of $30 \times 0.60 = 18.0$ sq. in. Two layers will be used at right angles, the distance between centers of layers being 1 in., which with 3 in. of insulation for the lower layer gives a total depth of footing of 20 in.

The maximum bond stress in the reinforcement is $171{,}000/(2.75 \times 30 \times 0.877 \times 16) = 148$ lb. per sq. in.

The diagonal tension is $352{,}000/(4 \times 5.17 \times 12 \times 0.877 \times 16) = 101.3$ lb. per sq. in.

It will be necessary to design a stirrup system for all diagonal tension in excess of 50 lb. per sq. in. If vertical $\frac{3}{8}$-in. round rods with prongs spaced at 6 in. are used (see section *AA*, Fig. 13-7*a* for arrangement), the strength of one line of stirrups in a 12-in. length will be $18{,}000 \times 0.11 \times 2 = 3{,}960$ lb. The distance between stirrup rings at the critical section will be $s = 3{,}960/(12 \times 51.3) = 6.0$ in. Letting x be the distance in feet from the center of the column to where stirrups will not be needed, namely, where the diagonal shear is 50 lb. per sq. in., we have

$$50 = \frac{3{,}000[\overline{12}^2 - (2x)^2]}{4 \times 2x \times 12 \times 0.877 \times 16} \qquad \text{or} \qquad x = 3.82 \text{ ft.}$$

We shall use six concentric rings of stirrups, the rings to be spaced at 6 in. centers, the first ring being under the sides of the column and the last ring 2.50 ft. from the column face.

13-9. Examples of Isolated Footings. Two forms of typical reinforced-concrete spread footings as used for the column foundations of a railway terminal station in Atlanta, Ga., are shown in Fig. 13-9*a*. The one illustrated on the left was used under 20- by 24-in. columns and is in the form of a truncated pyramidal slab reinforced with bars and stirrups. The one shown on the right is of the slab-and-beam type.

The 125-ft. concrete-block chimney for the St. Joseph's Home, Chicago, was founded on blue clay, the base of the foundation extending about 5 ft. below the surface of the ground. The footing, shown in Figs. 13-9*b* and 13-9*c*, consists of a circular slab 24 ft. in diameter and 10 in. thick, on which is built a box with a square outer surface 8 ft. 3 in. on a side and with an octagonal inner surface about 6 ft. 7 in. between opposite faces. There are two cantilever ribs extending from each corner of this box almost to the outer edge of the slab. These cantilever ribs are 14 in. wide at the box and 8 in. wide at the outside. Their reinforcement consists of 1-in. round bars and $\frac{1}{4}$-in. vertical stirrups. The base is thus made up

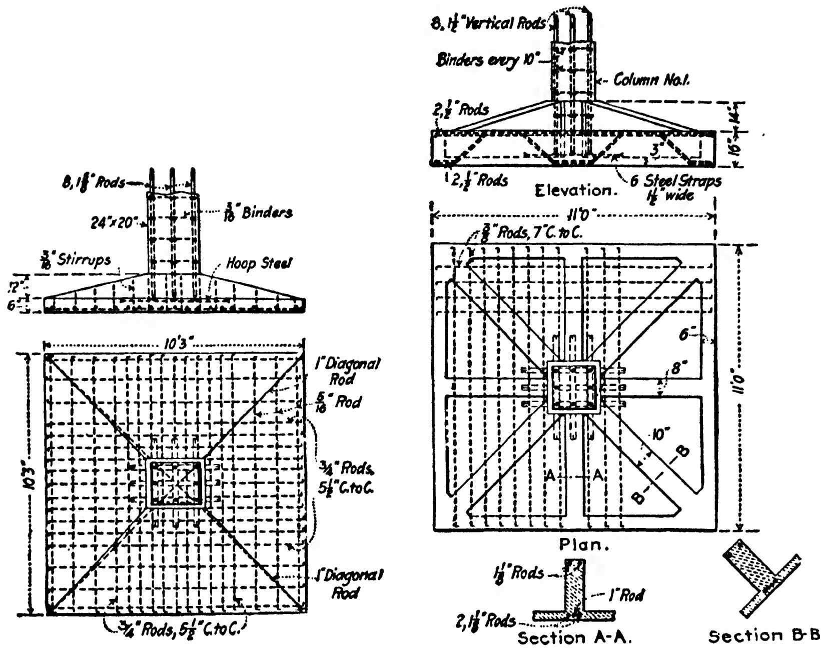


FIG. 13-9a.—Pyramidal and Ribbed Slab Footings of Reinforced Concrete, Atlanta Terminal Station, Southern Railway.

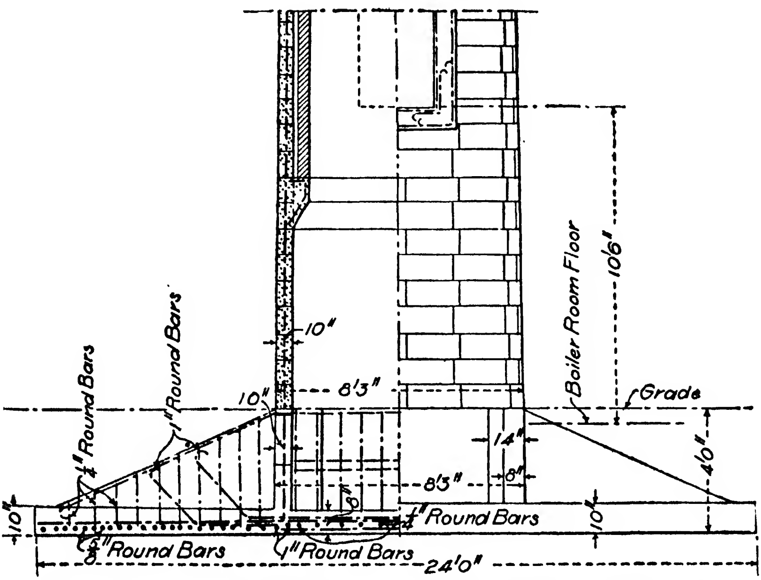


FIG. 13-9b.—Slab and Box Footing of Reinforced Concrete for a 125-ft. Chimney in Chicago.

of a series of slabs, each supported by the adjacent cantilever ribs and reinforced with $\frac{5}{8}$-in. round bars spaced according to the position of the slab in the base.

Perhaps the largest isolated reinforced-concrete spread footing ever used is that of the San Jacinto Monument[1] in Texas. The base is 124 ft. by 124 ft., the corners being beveled to give a bearing area of 15,040 sq. ft. to take the load of 37,200,000 lb. with a soil pressure of approximately 2,500 lb. per sq. ft. The thickness at the outer

Fig. 13-9c.—View of the Same Footing as Shown in Fig. 13-9b.

edges is 5 ft. and over a central area 48 ft. square is 15 ft. The two-way reinforcement consists of 2-in. square bars spaced $6\frac{1}{2}$ in. on centers.

13-10. Reinforced-concrete Mat Foundations. In the outline of the history of shallow spread foundations given in Art. 13-1 attention is called to the early type of masonry mat foundation which proved unsatisfactory. A type which has come into use resembles this early form in that the whole basement floor forms a mat but differs from it in that the concrete is heavily reinforced. This mat is designed as an inverted slab system spanning between supporting points and carrying the building weight as a load uniformly distributed over the soil.

[1] See *Civil Eng.*, vol. 7, p. 484, July, 1937.

Figure 13-10*a* illustrates the mat foundation used for a factory in Brooklyn, New York. It was of the slab-and-beam type, the slab thickness being 1 ft. and the beam thickness 3 ft. The beams formed continuous lines under the outer walls and along the center lines of the columns lengthwise of the building, the column spacing being 16 ft. 10 in. longitudinally and approximately $19\frac{1}{2}$ ft. transversely. The beams were 5 ft. wide under the walls and 6 ft. wide under the columns. As shown in the illustration the slab reinforcement consisted of 1-in. square bars. The beams under the columns were reinforced with eleven $1\frac{1}{4}$-in. square bars near the upper surface, the five center bars being carried through straight and the six outside bars bent down under the columns.

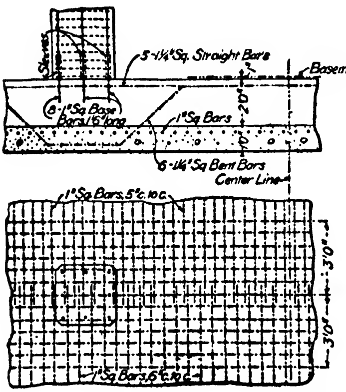


FIG. 13-10*a*.—Mat Foundation Composed of Slab and Beams.

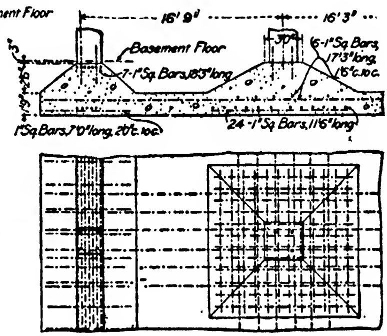


FIG. 13-10*b*.—Mat Foundation of Flat-slab Type.

Figure 13-10*b* illustrates another type used in the same city, which consisted of a 21-in. slab over the whole area surmounted by truncated pyramidal slabs under all columns and a trapezoidal-shaped slab under the walls. The columns were spaced approximately 16 ft. on centers in both directions. The column footings were raised $2\frac{1}{2}$ ft. above the raft slab. The reinforcement is clearly shown in the illustration.

In the foundation of the Pope Building, Cleveland, Ohio, a combination of a steel-grillage and a reinforced-concrete raft foundation was used. The material on which the foundation was placed consisted of a few feet of quicksand overlying clay. As the sides of the lot were enclosed by a permanent steel cofferdam extending well down into the clay, the quicksand was not subject to outside disturbance and hence made a satisfactory cushion. As shown in Fig. 13-10*c*, a 6-in. layer of concrete, well reinforced, was placed

on the waterproofing. The I-beam grillages, which were supported by the concrete slab, consisted of two tiers of 24-in. I-beams, each supporting a single column.

Where the bearing capacity of the soil is such that the footings are not required to cover the whole cellar area, the type illustrated in Fig. 13-10*d* may be used. This is a modern application of the inverted-arch type illustrated in Fig. 13-1*a*, reinforced concrete replacing brick. The arches ran in both directions between columns. They were 12 in. deep at the crown, 42 in. deep under the cast-iron column bases and varied from 4 to 5 ft. in width. The

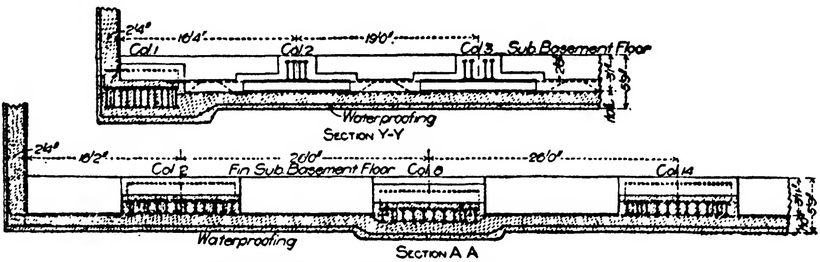


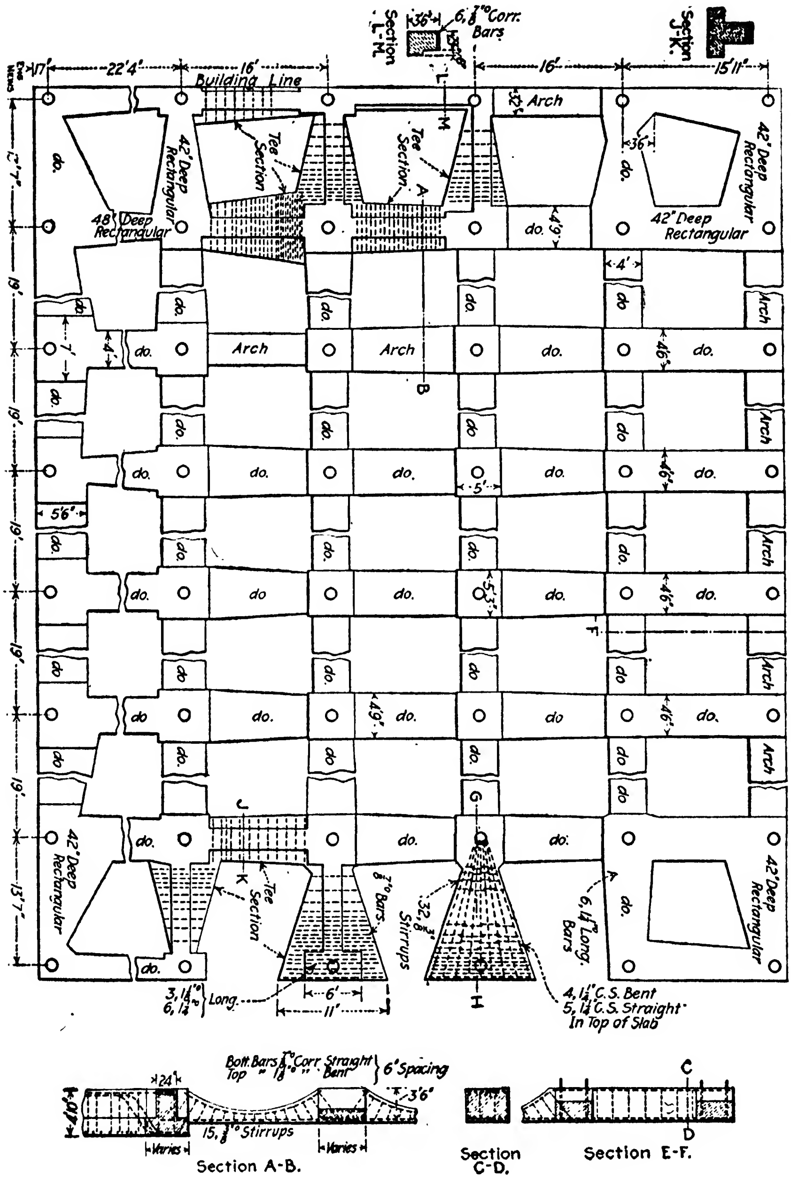
FIG. 13-10*c*.—Cellar-floor Sections Showing Grillage Beams and Reinforced-concrete Girders, Pope Building, Cleveland, Ohio.

reinforcement consisted of $\frac{7}{8}$-in. straight rods in the bottom, spaced 6 in. on centers, and $1\frac{1}{8}$-in. bent bars at the top, spaced the same distance. All end spans were made of rectangular or T-shaped reinforced-concrete beams, to provide for the thrust in the adjoining arches.

13-11. Rigid-frame Foundations. The mat foundation described in the preceding article cannot satisfactorily provide for large variations in column loads or span over large areas of soft soil. A recent development, which overcomes these limitations and also lessens the characteristic tendency of raft foundations to sag or dish at the center of the building, is to make a rigid-frame structure of the foundation mat and superimposed basement stories.

This type was first used in 1930 for a new section of the main telephone building[1] at Albany, N. Y. The building site was underlaid with a clay deposit over 100 ft. deep. A study indicated that this clay, which contained nearly 50 per cent by volume of water and was soft and plastic when worked, would support near the surface a load of from 2,000 to 3,000 lb. per sq. ft. with an ultimate settlement of about 2 in. The average load due to the weight of the building was 4,500 lb. per sq. ft. However, as the foundation

[1] *Eng. News-Record*, vol. 105, p. 836, Nov. 27, 1930.



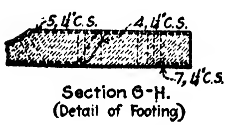


FIG. 13-10d.—Reinforced-concrete Arch Foundation of Warehouse at 418–426 West 25th Street, New York City.

mat was from 20 to 25 ft. below ground level, the general excavation removed a load of about 2,500 lb. per sq. ft. at the subgrade level, for which there could be substituted an equivalent weight of building without causing additional settlement of the underlying soil.

Figure 13-11*a* shows a cross section of this rigid-frame foundation. The foundation mat, together with the walls, columns, and floors of the basement, forms a rigid frame of the Vierendeel truss

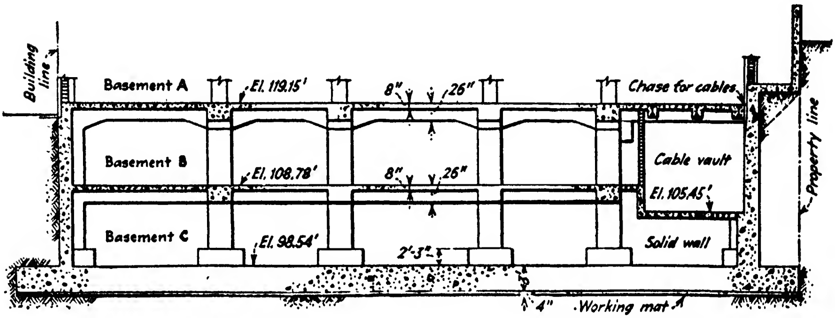


FIG. 13-11*a*.—Cross Section of Rigid-frame Foundation for Main Telephone Building in Albany, N. Y.

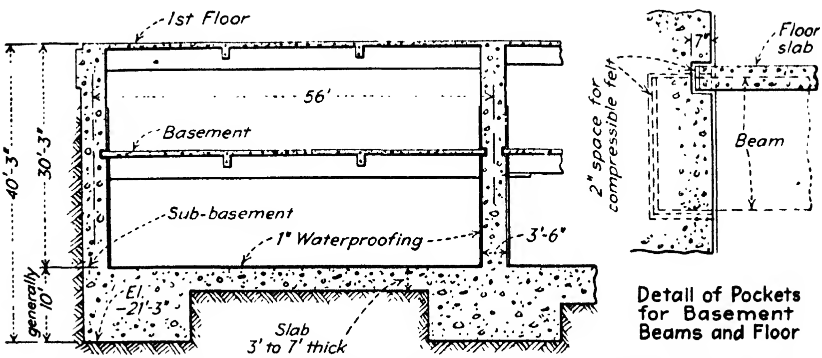


FIG. 13-11*b*.—Reinforced-concrete Box Substructure of New England Mutual Life Insurance Co. Building in Boston.

type. The floor systems of the *A* and *B* basements form the upper chord, the foundation mat the lower chord and the side walls and columns the verticals. Stresses were analyzed by the use of elastic models.

Another example of the use of the rigid-frame type is found in the 194- by 337-ft. office building of the New England Mutual Life Insurance Company in Boston. Bedrock at the site is overlain with 30 ft. of hardpan, some 70 ft. of glacial clay, about 20 ft. of silt up to mean tide level and about 20 ft. of man-made fill of sand and gravel. The footings rest on the clay, 40 ft. below ground

level, the upper 8 or 10 ft. of which is firm, oxidized soil, with softer, unoxidized clay below.

As shown in Fig. 13-11*b*, the reinforced-concrete box substructure was a monolith consisting of walls 40 ft. high, cast integral with wide continuous footings and with floor slabs, cast integral with the footings, completing the base.[1] There were four longitudinal and six cross walls, the minimum thickness being $3\frac{1}{2}$ ft. The footing slabs were from 15 to 18 ft. wide and from 8 to 10 ft. deep, while the floor slabs were from 3 to 7 ft. deep.

The walls were reinforced as girders, and the outer walls also as retaining walls for both vertical and horizontal transfer of earth pressure, the first to the footings and the second to the cross walls. The outer-wall footings were so proportioned that the active earth pressure on the walls brought the resultant pressure to the middle of the footing width. The footings were reinforced not only to take the stress resulting from the cantilever action, but horizontal steel was placed along the inner face also to enable them to function as horizontal girders to take the earth pressure to the cross walls.

Details were worked out so that the floor beams would not take any of the earth pressure thrust. This was done by providing expansion joints for the basement floor beams and by not placing the beams of the first floor until the backfill had been placed and thoroughly tamped.

Since the building was carried down to clay, the weight of the excavated material practically equaled the weight of the building, which was about 130,000 tons or 2 tons per sq. ft. of basement area. Based on the area of the wall footings the average pressure is 4 tons per sq. ft. without taking account of buoyancy or 3 tons per sq. ft. after deducting water uplift.

[1] *Eng. News-Record*, vol. 123, p. 692, Nov. 23, 1939.

CHAPTER XIV

BRIDGE PIERS

14-1. General Requirements. In selecting the site of a bridge and arranging the piers, careful attention must be given to such matters as location of crossing, position and spacing of piers and abutments, height of bridge, required waterway, etc. Where the construction is in new country, the location of the bridge can usually be made to suit the engineering requirements. These will be best satisfied where the width of the river is not great; however, it should not be located in the narrowest part, for there the current is apt to be swift and the water deep in heavy rains, thus making the construction of the substructure both difficult and expensive. On the other hand, where the bridge is located in a built-up community, it will have to be placed where it will best serve the needs of the people. If it is a highway structure, it will connect main thoroughfares on the two sides of the river; a railroad structure has to connect the rights of way. Building new streets or buying rights of way is very expensive in built-up vicinities and will usually be in excess of any possible saving in the cost of the bridge by placing the latter in a more advantageous position from an engineering standpoint.

In determining the number of piers and their spacing, due regard should be given to the financial considerations, the navigation interests, waterway requirements, and the Government rules and regulations.

The financial requirements are best served by an arrangement which makes the total cost of the bridge, superstructure plus substructure, a minimum. As the cost of the superstructure, exclusive of floor system, varies approximately as the square of the length of a span, and the cost of a pier with its foundation is approximately a constant for fairly wide ranges of span length, there is some length of span which, with corresponding number of piers, will make the total cost of the bridge a minimum. The total cost of a bridge having all spans of equal length may be roughly expressed by the equation

$$C = 2A + P\left(\frac{l}{x} - 1\right) + kl + \frac{l}{x} k_1 x^2,$$

where C = cost of bridge
A = cost of one abutment
P = cost of one pier, including its foundation
k = cost of floor system per foot of bridge
k_1x^2 = cost of one span of superstructure exclusive of floor system
x = length of one span
l = length of bridge

For minimum cost we shall place the first differential of this equation equal to zero

$$\frac{dC}{dx} = 0 - \frac{Pl}{x^2} + 0 + lk_1 = 0$$
$$P = k_1x^2.$$

Hence for maximum economy the cost of one pier including its foundation should equal the cost of one span of the superstructure exclusive of the floor system.

At many locations the question of furnishing adequate waterway is of primary importance. Where the bridge obstructs the free

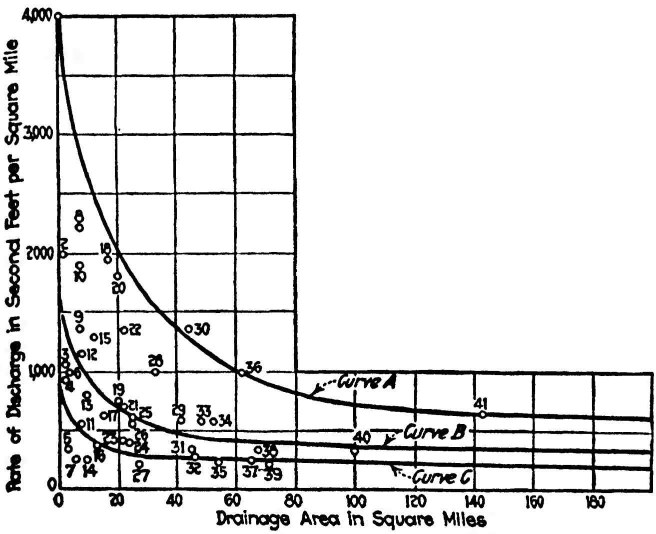


FIG. 14-1a.—Rates of Discharge from Small Drainage Areas.

flow of the stream, the elevation of the water above the bridge is raised (Art. 14-5), thus increasing probable flood damage and possibly endangering the safety of the structure itself. In Fig. 14-1*a* are shown actual flood flow records, the ordinates representing the rate of discharge in cubic feet per second per square mile and the

abscissas the drainage area in square miles. Ivan E. Houck[1] recommends using curves A, B, and C in designing waterways. Except where the channel cross section is unusually large or where damage resulting from floods or failure of the structure would be great, it is generally impracticable to design waterways as large as required by curve A. However, where possible, waterways should be designed to meet the requirements of curve B and in no case should values be provided for less than shown in curve C. A mean velocity of 10 ft. per sec. may safely be allowed under the bridge.

Navigation interests require that the piers shall be placed so as to cause as little danger and obstruction as possible to river traffic. Thus they should be kept out of the channel and should be spaced at considerable distances apart.

The pier should rest on a stable, unyielding foundation, the base of which is well below the frost line and below the elevation of any possible scouring action. Where rock or other satisfactory bearing material lies at a depth not greater than from 20 to 30 ft. below water level, the pier footing will usually be placed directly on the rock surface, a cofferdam being used if necessary. The material overlying the rock is first removed, after which the latter should be leveled or stepped off and cleared of all loose material before placing the footing for the pier.

For depths varying from 20 to 40 ft. or more a pile foundation may prove the cheapest. The correct principles of design for this type of foundation are discussed in preceding chapters. For depths greater than about 40 ft., some type of caisson foundation is often used.

Shallow foundations, corresponding to the spread footings so much used for buildings, are seldom used for bridges. Up to about 50 years ago, a spread footing consisting of a timber grillage was a common type of foundation for bridges. The grillage consisted of a more or less open mass of timbers laid directly on the gravel bottom after dredging out a few feet, and extending to nearly low-water level. The grillage was built, with courses alternating in direction, to a height of a few feet on shore, after which it was launched, completed, towed to the site, and sunk by filling the open spaces between the timbers with stones, etc. The disadvantage of this type of foundation lies in the fact that it is practically impossible to land the grillage perfectly level, owing to the great difficulty of preparing a level foundation bed. Another disadvantage lies in the danger of scour.

[1] *Eng. News-Record*, vol. 88, p. 1071, June 29, 1922.

14-2. Definitions. A bridge pier is a structure, usually of concrete or of stonemasonry, which is used to transmit loads from the bridge superstructure to the foundation.

Some of the common parts of a bridge pier are the following:

Bridge Seat. A block of stone or concrete resting on top of the pier to support a pedestal or base plate.

Coping. The top course of the pier, usually projecting beyond the lower courses.

Belting Course. The course immediately below the coping, having an offset less than the coping.

Footing Courses. Those courses at or near the bottom of the pier, which are wider than those in the main part of the pier.

Body. The main part of the pier.

Starling. That part of the pier below high water, the horizontal section of which lies outside of the largest rectangle that can be formed on the two sides of the pier.

Starling Coping. The offset course at about high water which forms the top course of the starling.

Batter. The slope of the sides and ends of the pier.

The coping course serves to protect the pier from the weather. If made of stonemasonry, the stone is of the best quality and cut to make small joints; if of concrete, a rich mixture is employed. The top is usually made with a surface sloping from the middle downward to the sides and is often waterproofed with some waterproofing compound, especially when of concrete. It is customary to give the coping course an offset of from 6 to 12 in. in order to prevent rainwater from dripping down the sides and ends of the pier and also to improve the appearance of the pier.

The chief function of the belting course is to strengthen the coping offset, but it also improves the appearance of the pier. In special cases two or three belting courses are used; at other times none are employed.

The function of the footing course is to distribute the load over a larger area than the base of the body of the pier. Unless reinforced, the slope of the footing should not be over 30 deg. with the vertical; where reinforced, the slope may be anything consistent with safe stresses in the steel and concrete as determined when considering the projecting footing courses to act as a cantilever beam.

As explained in Art. 14-3, the function of the starling is to pass the water with the least possible disturbance, for then there will be the least pressure against the pier due to current, ice, and

drift, less danger to navigation from eddies, and less danger from underscouring.

14-3. Form, Dimensions, and Quantities. The two primary requirements of bridge piers are (*a*) to transmit the load from the superstructure to the foundation and (*b*) to disturb the natural movement of the water as little as possible. Naturally, a minimum capitalized cost should also be sought. As the load from the superstructure is generally applied on the pier at essentially two points, at a distance apart equal to the distance between trusses, the most economical way of satisfying the first requirement is to use two cylinders, one under each load, as described in Art. 15-1.

On the other hand, the second requirement is best served with a form resembling a ship, modified to increase the stability of the pier against floating ice, debris, etc., and to make the construction cheaper. The shape generally used is that of a rectangle with triangles or segments of circles at both upstream and downstream ends, or at only the upstream end. The advantage of having starlings at both ends is that the foundation becomes symmetrical with the vertical loads on the pier, thus avoiding an uneven distribution on the foundation bed due to these loads; eddying at the downstream end of the pier is also reduced. Starlings are necessary only below high-water level. When used in swift streams filled with ice in winter, the starlings are heavily reinforced with old rails or structural shapes (see Art. 14-6).

Where segments of circles are used for starlings, the curves are tangent to the sides of the pier. A semicircular shape is often employed, but in other cases the radius is made somewhat greater than half the pier thickness, to give a pointed nose. A value used on many piers and recommended by G. S. Morison is three-quarters the width of the pier. Above high water the pier ends may be made square, but a much better appearance is secured when a semicircular form is used. A combination of straight and circular nose is sometimes used (Art. 14-7).

The dimensions of bridge piers depend on the load to be supported, class of superstructure, height of pier, type of foundation, and magnitude of lateral forces to be resisted.

The dimensions of the top of the pier depend on the distance between trusses or girders, plus a certain amount necessary to prevent the load from the pedestals approaching too closely the edges of the pier under the coping. It is often specified that the width shall not be less than 4 ft., or less than that required for the bearings of the superstructure plus 1 ft., or less than that necessary to give the required stability. Stability is discussed in Art. 14-9.

It is also specified that the length under the coping shall not be less than the distance out to out of superstructure bearings plus $1\frac{1}{4}$ times the width of the pier.

The coping course usually has a depth varying from 1 to $2\frac{1}{2}$ ft., depending on the general dimensions of the pier. The depth should not be less than one-sixth of the thickness of the stem of the pier measured under the coping. Copings should project over the faces of the stem a distance equal to about one-third the depth. This may be increased where belting courses are used. In the Thebes bridge piers the thickness of the stonemasonry coping was 27 in. and the projection 24 in. This large projection was made possible by the use of a heavy belting course as shown in Fig. 14-6*d*.

Where a belting course is used, it is usually made of about the same thickness as the coping course, and its projection beyond the stem of the pier is closely equal to the projection of the coping course beyond the belting course. As to whether the belting course will be double or single will depend on the desired total offset. In the following articles are a number of illustrations showing arrangements of the belting course.

The sides of the body or stem of the pier are usually given a batter of 1 in 24 or 1 in 12. The lesser batter is more commonly used for high piers and the greater for low piers.

The footing courses serve to transfer the load from the body of the pier to the foundation. The upper surface of the upper footing course should project only about 1 ft. beyond the face of the stem of the pier. The depth of any footing course should not be less than

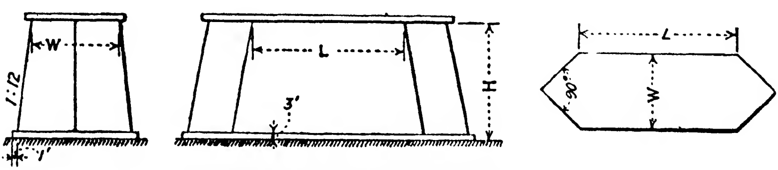


Fig. 14-3*a*.—Outline of Standard Concrete Pier.

2 ft., and the courses may be stepped off at an angle of about 30 deg. with the vertical or have a uniform batter of the same amount. When constructed on pile foundations, the footings should extend at least 6 in. down over the pile tops, and the distance from the outside face of the footing to the center of any pile should not be less than $1\frac{1}{2}$ ft.

Figure 14-3*a* shows a form of pier having triangular ends, the volume of masonry for which is given in Fig. 14-3*b* for various combinations of height, length, and width.

Table 14-3*a* gives the quantities for railroad bridge piers[1] having square or semicircular ends, for widths of 6 and 8 ft.. lengths from 16 to 37 ft., and heights from 8 to 50 ft.

TABLE 14-3*a*

CUBIC YARDS OF MASONRY IN PIERS.

Height "H"	W×L	Square Ends								Round Ends							
↓	→	6×16	6×29	6×24	6×37	8×16	8×29	8×24	8×37	6×14	6×27	6×22	6×35	8×14	8×27	8×22	8×35
8		60	105	88	133	77	135	113	170	72	117	100	145	99	157	136	194
14		88	153	128	194	112	197	164	249	104	170	145	210	143	227	196	280
20		118	206	172	264	151	263	220	352	138	227	193	280	190	302	260	372
26		151	263	220	331	192	333	279	421	176	288	245	357	240	382	328	470
32		187	323	271	409	236	408	342	513	217	353	300	437	293	456	400	573
38		226	389	327	490	283	488	410	613	260	423	360	524	350	555	478	681
44		268	459	385	577	334	572	481	718	307	496	424	615	411	650	560	798
50		313	534	450	670	388	662	558	831	356	577	492	712	474	748	644	917

Quantities include 5 ft. foundation as shown in sketch.

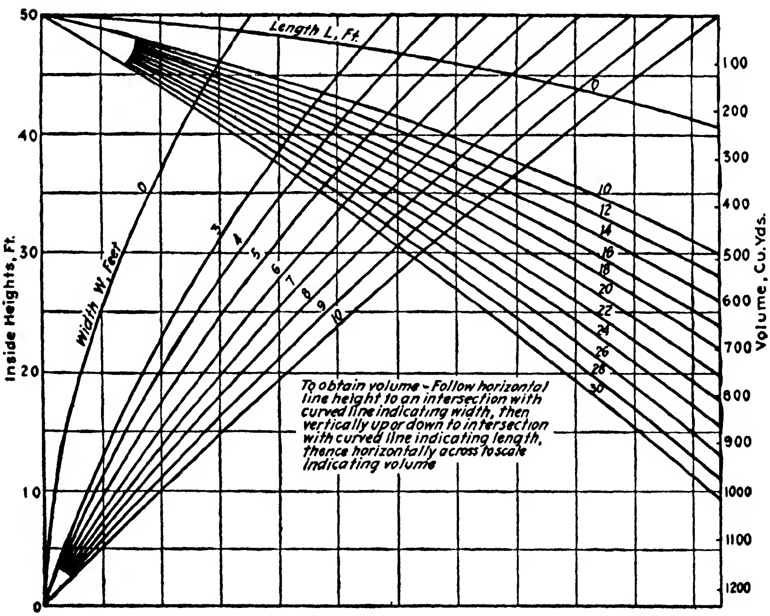


FIG. 14-3*b*.—Diagram for Cubature of Concrete Piers.

Figures 14-3*c* and 14-3*d* give valuable data[2] for estimating quantities in piers of highway bridges with semicircular ends, the

[1] From Railway Engineering and Maintenance Cyclopedia, 1926.

[2] McCullough, C. B., "Economics of Highway Bridge Types," Gillette Publishing Company.

first being for piers with a batter of ½ in 12 in. and the second for a batter of 1 in 12 in.

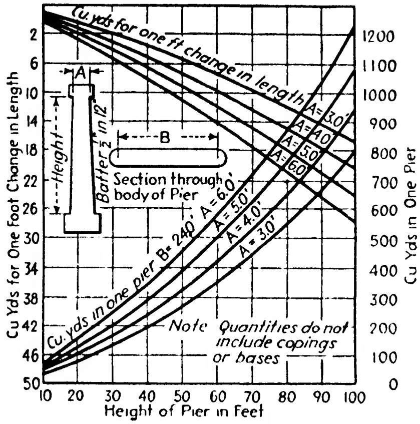


FIG. 14-3c.—Quantities in Highway Bridge Piers Having a Batter of ½ in. in 12 in.

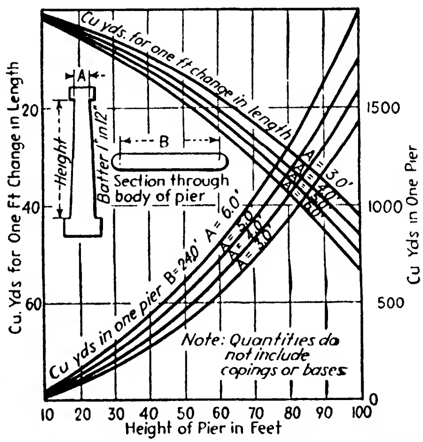


FIG. 14-3d.—Quantities in Highway Bridge Piers Having a Batter of 1 in. in 12 in.

14-4. Materials and Construction. Previous to about 1880 it was the universal rule to build piers entirely of stonemasonry, whereas at the present time most piers are built either entirely of concrete or of a concrete hearting and stone facing. The three following conditions have brought about this change: (*a*) the decrease in the cost of cement, (*b*) the increase in the strength and the greater reliability of cement and concrete, and (*c*) the increased cost of cut stone, due to the labor factor. Piers are sometimes built using a cluster of steel piles (Art. 7-3). For small and temporary structures, timber may be used (Art. 14-8).

Among the earliest of the all-concrete piers in this country were those used for a bridge across the Medina River, 18½ miles west of San Antonio, Tex., built in 1881. In Nova Scotia they were first used in 1883. In both of these instances concrete was used because of the absence of good stone in the vicinity and the high cost of transportation. In Europe the all-concrete pier was used somewhat earlier than the above dates. For some years after its introduction the development of the all-concrete pier was slow. In an address delivered in 1899, G. S. Morison[1] said:

> Prejudices have been raised against it [concrete] through inferior work done in this country when it was first introduced, but it is within the limits of possibilities that an artificial stone can be made in this way which will be as good and as durable as the natural stones which are commonly used; when this is accomplished the advantages of a truly monolithic construction will make concrete the best building material, and, except for the facings of monumental works, where nothing can take the place of the finest stone from nature's laboratory, it may be universally used.

In the stonemasonry pier as exemplified in many large bridges built by G. S. Morison, the facing courses are mostly limestone ashlar, with granite ashlar for the upstream nose stones for all courses between high and low water. The backing is composed of limestone rubble, in some cases with, and in other cases without, coursed joints. For the Bellefontaine bridge, built in 1892, it was specified that the backing stones should have the same thickness as the face stones, that the spaces between the large stones of the backing should not occupy more than one-fifth of the volume of the pier inside the face stones, and that these spaces should be filled with good rubble masonry.

The piers for the Merchant's bridge across the Mississippi River at St. Louis, built in 1889, were among the early large piers to have concrete backing. Here the coping course, the three courses below this, and the starling coping course were all of stonemasonry, the remainder of the backing being concrete. Figure 14-4*a* shows the details of the stonemasonry for the starling coping for piers I and IV.

For complete and up-to-date specifications for stonemasonry the reader is referred to the Manual of the American Railway Engineering Association.

The advantage of concrete over stonemasonry lies in its lesser cost. Although its compressive strength is somewhat less than that of first-class stonemasonry, yet, on account of its monolithic char-

[1] *Eng. Record*, vol. 39, p. 497, Apr. 29, 1899.

acter, most engineers agree that it is the more suitable material, except possibly for the facing of the pier. Concrete mixtures approximating 1:2½:5 are usually used for mass concrete in piers, the coping course being somewhat richer and approximating a 1:2:4 mixture.

There are some advantages, however, in using a facing of stonemasonry, among these being the saving in the expense of forms, the more rapid rate of construction possible, the more attractive appearance of the pier, and the elimination of surface cracks. These surface cracks, almost always present in plain concrete piers, are due

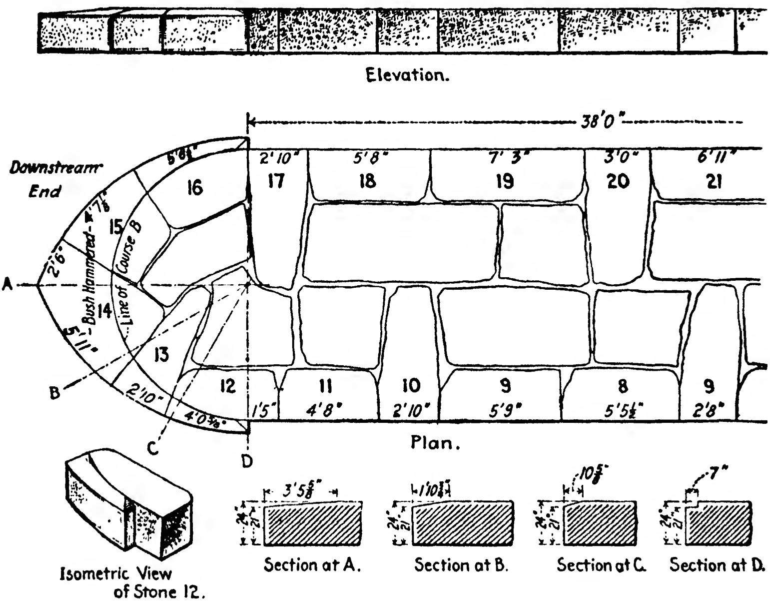


Fig. 14-4*a*.—Starling Coping Course for Pier IV, Merchant's Bridge, St. Louis.

to the expansion and contraction, caused by temperature changes, of the outer layer of concrete.

Where a stonemasonry facing and concrete backing are used for piers bearing very heavy loads, the facing stones should be tied in with rods, as shown in Fig. 14-6*c*. Where the all-concrete pier is used, it is advisable to place reinforcing rods near the surface. This reinforcement will prevent the occurrence of, or at least decrease the size of, the cracks noted above and will also add an element of safety by taking any tensile stresses in the concrete. Reinforcement in horizontal planes under the coping and above the bottom of the footing serves to carry the loads more uniformly into the pier and foundation.

A typical specification requires that all faces of the stem above the footing courses shall have surface reinforcement consisting of a network of vertical and horizontal bars embedded in the concrete about 2 in., the weight being about $2\frac{1}{4}$ lb. per sq. ft. of surface for railroad structures and $1\frac{1}{2}$ lb. for highway structures. The faces of the coping course should also be reinforced to about the same degree. Where pile foundations are used, the lower footing course should have a horizontal network of reinforcement placed about 6 in. above the pile tops and weighing about 3 lb. per sq. ft. of horizontal section of the footing course for railroad structures and 2 lb. for highway structures. The stem of the pier should have similar horizontal layers near the top and bottom and at intervals not exceeding 20 ft. A similar network should be embedded in the coping about 2 in. below its upper surface.

14-5. Obstruction of Piers to Flow of Water. The presence of piers in a stream compels the water to flow through a reduced cross section; hence the water must acquire a higher velocity in passing the piers than obtains in the unobstructed channel. This obstructing of the natural flow results in a permanent raising of the water level above the bridge. The amount of rise measures the head of water lost by the presence of the piers and depends on (*a*) the friction of the water on the pier walls, (*b*) the contraction of flow on entering the restricted section, and (*c*) the expansion of the stream on leaving the piers. Experiments indicate that the loss of head caused by friction along the pier walls is small compared with the other two effects. The contraction of the stream at the upstream nose of the pier causes a disturbance in the flow, resulting in eddies and cross currents as well as increased velocity, the amount of disturbance depending on the shape of the nose. As the water leaves the downstream nose, a second disturbance results.

The amount of contraction, and consequent disturbance of natural stream flow, depends not only on the shape of the pier but also on the quantity of water flowing and the reduction of waterway area resulting from the presence of the piers. Other factors which also have some effect are the lengths of the piers and the obliquity of position or angle which the axis of the pier makes with the direction of the thread of the stream.

For the results of tests of small models of piers the reader is referred to *Technical Bulletin* 442 of the U. S. Department of Agriculture entitled Bridge Piers as Channel Obstructions. This bulletin was written by David L. Yarnell and was published in November, 1934. Most of the tests were made in a channel 10 ft.

wide, the width of most of the piers being 14 in. The results of an earlier set of tests made by Floyd A. Nagler appear in a paper entitled Obstruction of Bridge Piers to Flow of Water, *Transactions of the American Society of Civil Engineering*, vol. 82, p. 334, December, 1918. These tests were made in a channel 2.138 ft. wide, the piers being 6 in. wide. Acknowledgment is made to the authors of these papers for the material in this article.

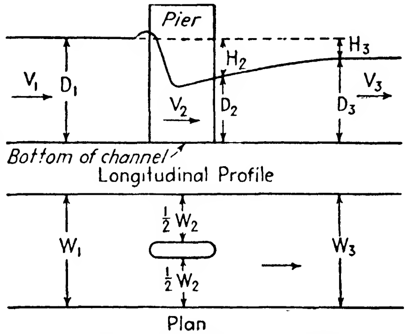


FIG. 14-5*a*.—Diagram Showing Symbols Used in Formulas.

Figure 14-5*a* shows a bridge pier with the water flowing through the contracted section. Prior to placing the pier, the depth of water all along the channel was D_3. The presence of the pier results in increasing this depth above the pier by an amount H_3. Nagler has proposed the following formula—see above noted references for limitations—for the relationship between the quantity of water flowing and the headwater:

$$Q = KW_2\sqrt{2g}\left(D_3 - \frac{0.3V_3^2}{2g}\right)\sqrt{H_3 + \frac{\beta V_1^2}{2g}} \qquad (14\text{-}5a)$$

where Q = quantity of water flowing in cubic feet per second
W_2 = mean width of stream in feet at the piers
D_3 = mean depth of the water in feet below the contracted section
V_1 = mean velocity of the water in feet per second above the contracted section
V_3 = mean velocity of the water in feet per second below the contracted section
H_3 = headwater in feet caused by the presence of the piers, that is, the increase in depth of water above the contracted section
g = acceleration in feet per second per second (32.2) due to gravity.
K = a coefficient depending mainly on the shape of the pier and the percentage of channel contraction
β = a coefficient varying with the percentage of channel contraction (see Fig. 14-5*b*)

Yarnell and Nagler made tests on a number of different shapes of piers, including those with square ends, semicircular ends, convex

ends (curves tangent to sides of pier and described on an equilateral triangle), lens-shaped ends (curves tangent to sides with a radius of twice the pier width), triangular ends with nose angles of 53, 60, and 90 deg., and piers composed of two shafts, with and without connecting diaphragms. These are graphically shown in Fig. 14-5c, the values of K appearing beneath the illustrations, 100α

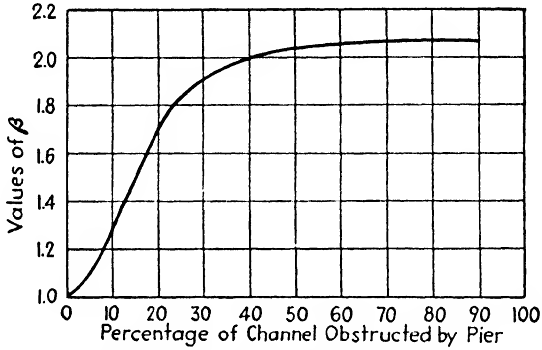


FIG. 14-5b.—Values of β in Eq. 14-5a.

representing the percentage contraction, L the length of the pier, and W the width. Symbols for the types of ends are as follows: S, square; R, round; C, convex; L, lens shaped; T, cylindrical; and P, pointed. Most of the piers had no batter; however, the type marked $P_{90°B}P_{90°B}$ had a nose batter of 1:12. Those marked NC,

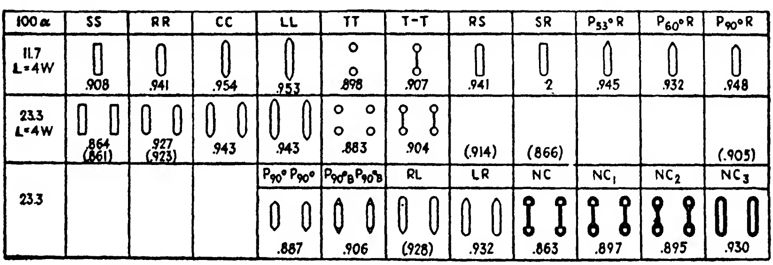


FIG. 14-5c.—Values of K in Eq. 14-5a for Different Pier Types.

NC_1, and NC_2 had pier nose and tail batters of 1:24, while that marked NC_3 had a 1:24 batter all around.

Tests made by Rehbock in Germany indicate that the flow of water past piers may be divided into three classes as follows: (a) ordinary flow, in which the water passes the obstruction with little or no turbulence; (b) intermediate flow, in which there is moderate turbulence; and (c) heavy flow, in which the turbulence is large.

The moving water will be in the first class as long as α is less than $\frac{1}{0.97 + 21w} - 0.13$, and in the third class when α is greater than $0.05 + (0.9 - 2.5w)^2$, where $w = \frac{V_3^2/2g}{D_3}$. It will be in the second class when α lies between these values. In Fig. 14-5c the values of K are for the first class of flow only, most problems falling in this class. For coefficients for the other two classes the reader is referred to the first of the two citations noted in this article.

Equation 14-5a shows that the value of the headwater H_3 will increase rapidly as the coefficient K is decreased; hence a high value of K indicates an efficient shape of pier. From this it follows that curved and pointed ends are better than square ends, especially with reference to the upstream end. As might be expected, cylinder piers offer more resistance to flow of water than do solid piers with curved or pointed ends.

In the Yarnell and Nagler experiments most of the tests were made with channel contractions of 11.7 and 23.3 per cent, the first using a single pier and the second two piers. However, Yarnell made a few tests for single-pier obstructions of 35 and 50 per cent. For piers with square ends he found coefficients of 0.867 and 0.888, respectively; and for piers with semicircular ends, 0.986 and 1.108, respectively.

A few tests were made by Yarnell on piers with semicircular ends to determine the effect of placing the long axis of the pier at an angle with the thread of the stream. For an angle of 10 deg. the coefficient was 0.936, and for an angle of 20 deg. it was 0.876, as compared with 0.941 for a zero angle. Thus it appears that a skew of 10 deg. increases the obstruction effect but slightly, but increasing this to 20 deg. has a pronounced effect. Yarnell also found that increasing the length of the pier from four to thirteen times its width had comparatively little effect on the hydraulic efficiency.

EXAMPLE. Let it be required to determine the probable heading up of a stream for the following data, the piers having semicircular ends:

Quantity of flow (Q) = 45,000 cu. ft. per sec.
Total cross-sectional area of flow = 7,560 sq. ft.
Cross-sectional area of piers = 720 sq. ft.
Width of flood area = 390 ft.
Width of four piers = 40 ft.

$$W_2 = 390 - 40 = 350 \text{ ft.}$$

$$D_3 = \frac{7.560}{390} = 19.4 \text{ ft.}$$

$$V_3 = \frac{45{,}000}{7{,}560} = 5.95 \text{ ft. per sec.}$$

$$\alpha = \frac{720}{7{,}560} = 0.0952$$

From Fig. 14-5*b* the value of β is found to be 1.24.

To determine the class of flow, $w = \dfrac{V_3^2/2g}{D_3} = \dfrac{\overline{5.95}^2/64.4}{19.4} = 0.0284$, and the limiting value of α for class 1 flow is

$$\frac{1}{0.97 + 21w} - 0.13 = \frac{1}{0.97 + 21 \times 0.0284} - 0.13 = 0.51;$$

hence our problem falls under class 1 flow. From Fig. 14-5*c* we see that the value of K will be slightly more than 0.941, say 0.944, since $\alpha = 0.0951$ is less than 0.117, the value for which $K = 0.941$. Assuming that the heading up will amount to 0.1 ft., we find that $V_1 = \dfrac{5.95 \times 19.4}{19.4 + 0.1} = 5.92$ ft. per sec.

Applying Eq. 14-5*a*, $45{,}000 = 0.944 \times 350 \times 8.02 \left(19.4 - 0.3 \times \dfrac{\overline{5.95}^2}{64.4}\right) \sqrt{H_3 + 1.24 \times \dfrac{\overline{5.92}^2}{64.4}}$ or $H_3 = 0.10$ ft. If square ends had been used with a value of $K = 0.915$, we would find that $H_3 = 0.14$ ft., or an increase of 40 per cent with a decrease in coefficient of 3 per cent.

14-6. Examples of Solid Piers. Figure 14-6*a* illustrates a simple form of the solid all-concrete pier used by the Western Maryland Railway. The dimensions are given in the diagram. The upstream end of the pier is built with its sides at a 45-deg. angle with its transverse axis to form a cutwater end, the nose of which extends 3 ft. 3 in. beyond the corner of the pier at the lower edge of the coping. This nose was molded to a circle by inserting within the forms a strip of No. 16 iron, 9 in. wide, bent to a 6-in. radius. It is held in place by 1-in. bolts, 9 in. long, extending into the concrete. They have a welded head on the end outside the plate and a head and a 2-in. washer on the end in the concrete.

A good example of the all-concrete pier with reinforcement near the outer surface is shown in Fig. 14-6*b*, which illustrates one of the piers for the Gilbertsville bridge. The bottom of the footing and the top of the coping are also reinforced.

Figure 14-6*c* shows the sectional elevation and plans of various courses of the part above high water of pier 3 of the Beaver bridge of the Pittsburgh and Lake Erie Railway. The facing is of ashlar sandstone with 1:3:5 concrete backing. As shown in the illus-

trations, the facing was securely tied to the backing by 1¼-in. rods running both lengthwise and crosswise. Extra rods were used to reinforce the hearting. Contrary to the usual practice in large stone-faced piers, a stone-coping course was not used, a ring around the outside being of stone and the rest concrete. The shoe grillage of I-beams which takes the load from the superstructure and

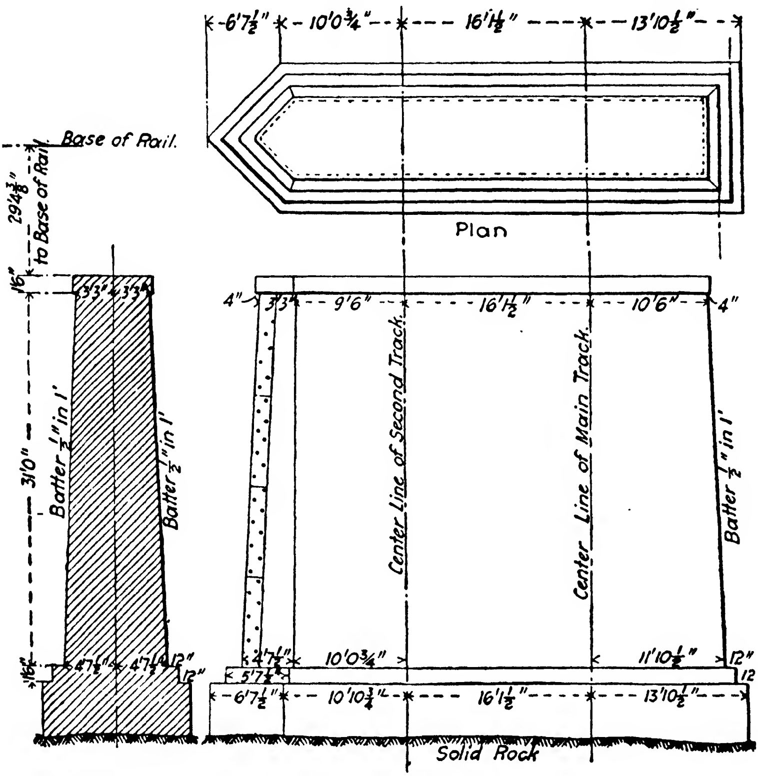


FIG. 14-6a.—Concrete Bridge Pier, Fourth Crossing of Potomac River, Western Maryland Railway.

distributes it over an area of 240 sq. ft. on the pier rests on and is supported by a 1:2:4 mixture of concrete (the darker portion in the illustration). To waterproof the top of the pier, a granolithic roof about 3 in. thick was placed over the entire top. The total load from the superstructure is 12,000 tons, and the pressure on the masonry under the grillage is about 25 tons per sq. ft.

Pier 2 of the Thebes bridge of the Illinois Central Railroad is a type of pier used in many large structures across the Mississippi

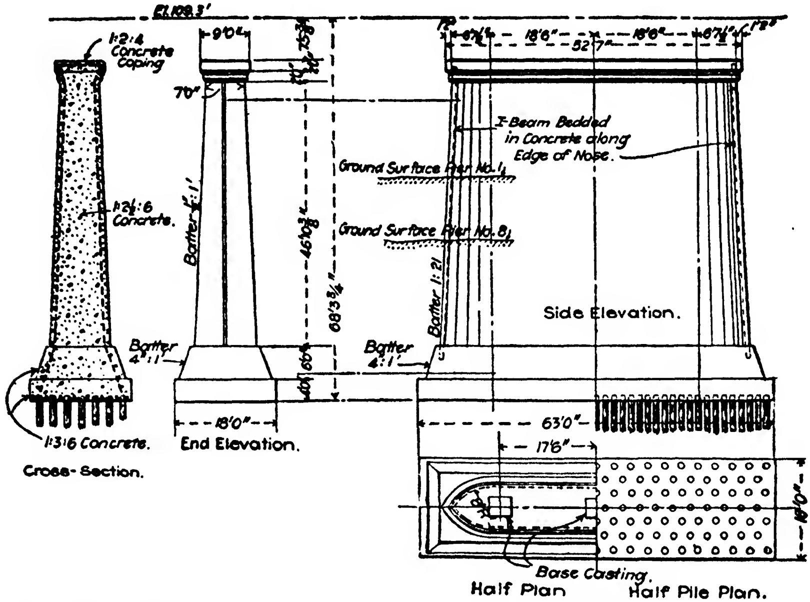


FIG. 14-6b.—General Dimensions of Piers of Illinois Central Railroad Bridge over Tennessee River, Gilbertsville, Ky.

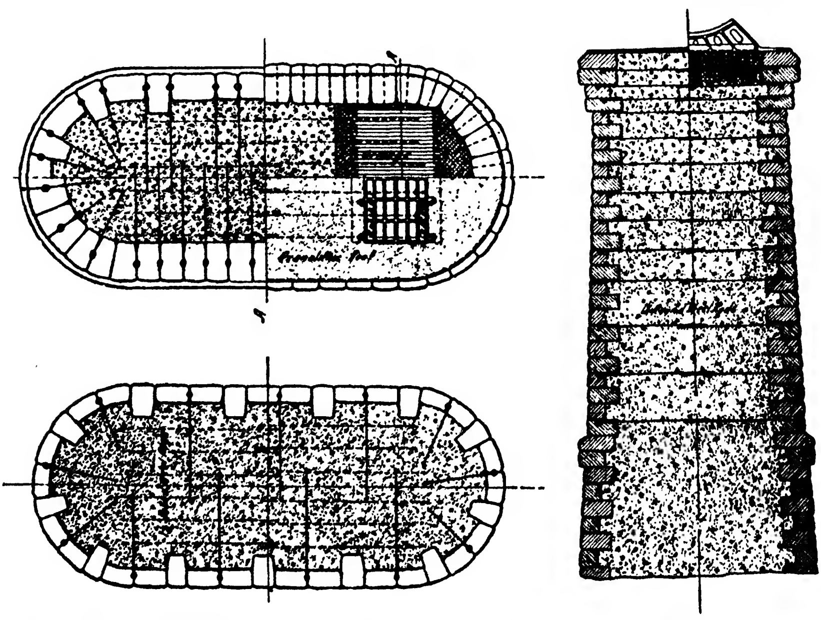


FIG. 14-6c.—Cross Section and Plans of Pier 3, Pittsburgh and Lake Erie Railway Bridge over Ohio River, Beaver, Pa.

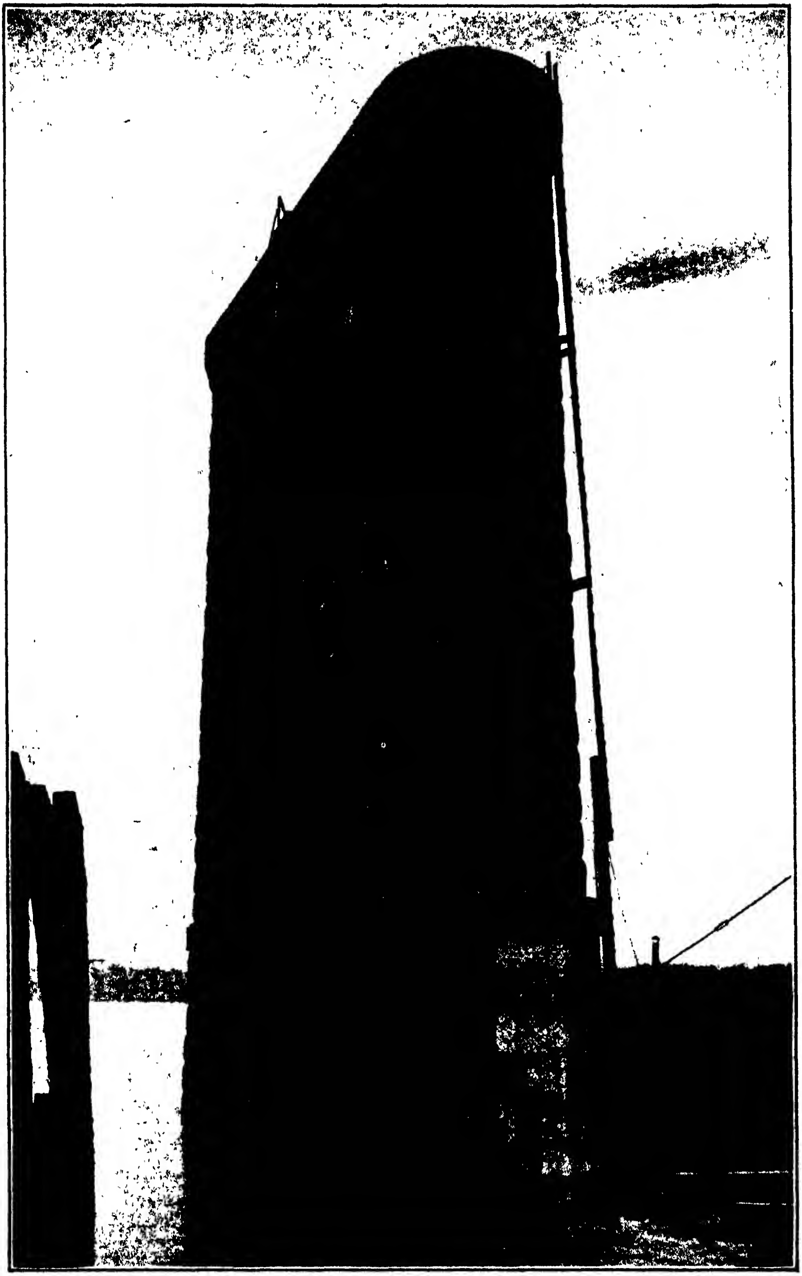


Fig. 14-6d.—Pier 2 of Cantilever Bridge over the Mississippi River at Thebes, Ill. Designed by Noble and Modjeski. April 1, 1905.

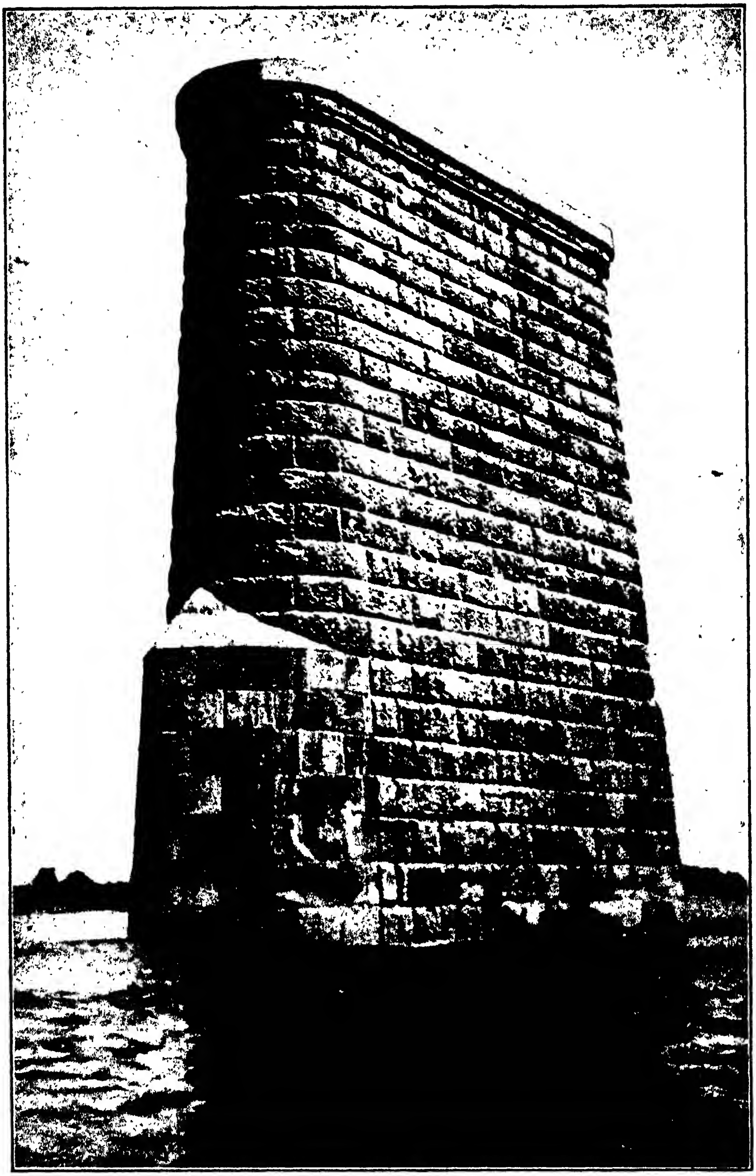


Fig. 14-6e.—Pier 3 of McKinley Bridge over the Mississippi River at St. Louis, Mo., Showing Starling with Conical Top. May 16, 1909.

and Missouri rivers. As shown in Fig. 14-6*d*, it is a very simple form of pier, and in its simplicity lies its beauty. In plan the sides are parallel and the ends are formed by two circular arcs meeting. Above high water the ends are semicircular. The coping projects 2 ft. beyond the pier, and the projection is divided between the

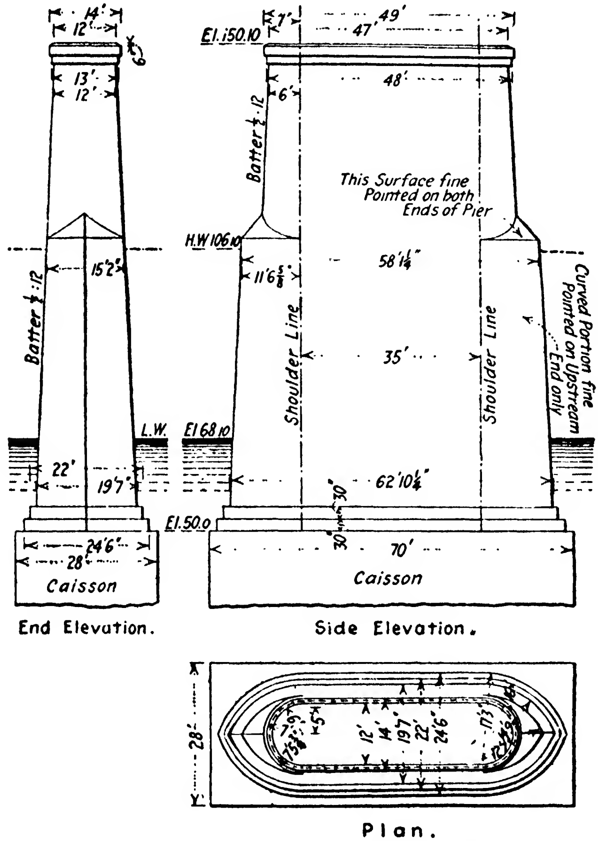


FIG. 14-6*f*.—General Dimensions of Pier 3 of the McKinley Bridge.

coping and the belting course below. The starling coping covers the starling only. The pier has a batter of 1 in 24.

Another bridge having piers of about the same form as that just described is the McKinley bridge at St. Louis. The most notable difference between the piers of the McKinley and Thebes bridges is in the treatment of the top of the starling. As shown in Figs. 14-6*e* and 14-6*f*, the starling coping in the former bridge is dispensed with and the top of the starling finished with a conical surface.

For the McKinley bridge piers the facing is of limestone, with the exception of the bridge seats and the upstream nose stones above the river bed, which are of granite. The hearting is of concrete, with the exception of the three courses below the coping, which are backed with limestone masonry.

The curved surfaces of the upstream starlings are close pointed to ¼-in. projection. The exposed surfaces of the main copings and the projecting bottom beds of the belting courses are planed. A 4-in. draft line is cut along the lower edges of the belting courses and on each side of the vertical angles of the downstream starlings. All other stones are quarry-faced, with projections not exceeding 3 in.[1]

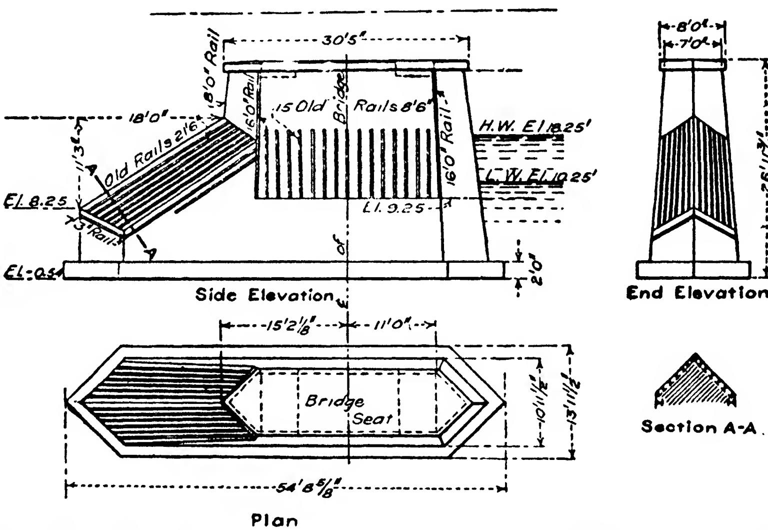


FIG. 14-6g.—Pier with Ice-breaking Cutwater. Flag Point Bridge of Copper River and Northwestern Railway, Alaska.

The piers of the McKinley bridge were designed by Ralph Modjeski and those of the Thebes bridge by Alfred Noble and Ralph Modjeski. The piers for both of these bridges resemble closely the standard type designed by George S. Morison.

Figure 14-6g illustrates the pier of a bridge across Copper River, Alaska, built to withstand very heavy ice pressure. The cutwater has a heavy slope to lift as well as to cut and divert the ice and is heavily reinforced with old track rails. The sides are also reinforced with rails.

Figure 14-6h illustrates the steel-plate protection for the nose of a pier of the Spokane bridge of the Inland Empire System. The

[1] *Eng. News*, vol. 63, p. 9, Jan. 6, 1910.

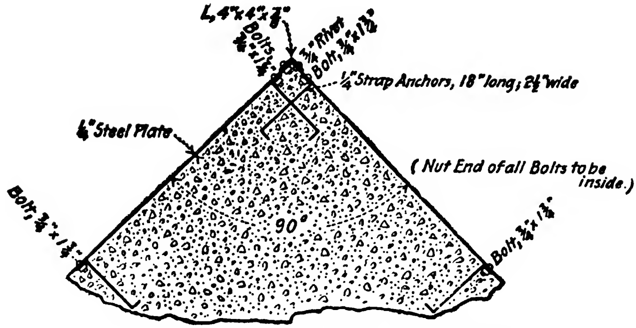


FIG. 14-6h.—Section of Steel Nose of Pier.

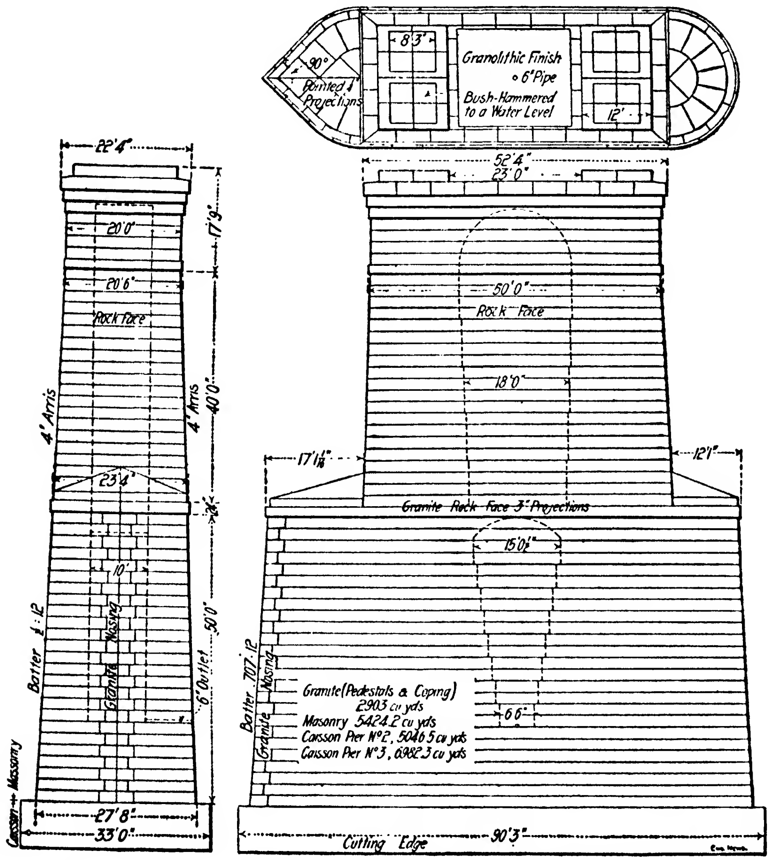
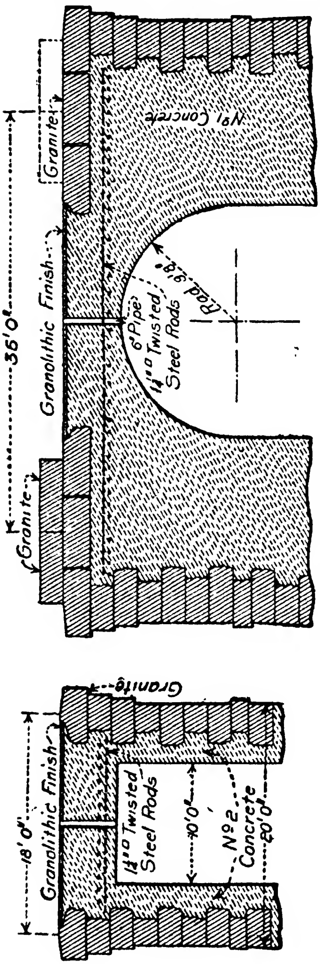


FIG. 14-7a.—Channel Piers of the Municipal Bridge over the Mississippi River, St. Louis, Mo.

steel plates were $\frac{1}{4}$ in. thick, 5 ft. $8\frac{1}{2}$ in. wide on each side of the vertex, and extended from the river bottom to above high water. They were anchored to the pier by $\frac{1}{4}$-in. Z-shaped straps 18 in. long and spaced 18 in. apart, staggered on the nose. At the vertex the plates were reinforced with a 4- by 4- by $\frac{3}{8}$-in. angle.



Sections on Center Lines of Pier

FIG. 14-7b.—Chamber in Upper Part of Piers 2 and 3, Municipal Bridge.

In Arts. 9-3, 9-10, and 9-11 are more illustrations of solid piers.

The world's tallest bridge pier is 358 ft. high and has a base 90 by 95 ft. in plan. It forms a part of a combined highway and railway bridge over the Pitt River arm of the Shasta Dam reservoir in California.

14-7. Examples of Hollow Piers. In the solid bridge pier a considerable part of the hearting near the top of the pier and between the pedestal bearings takes but little load. In other words, the pier acts more or less like a double-cylinder pier, the part directly under the bearings acting somewhat as independent legs to carry the load, the remainder acting chiefly as a bracing system. For this reason a considerable amount of concrete may be saved with but small loss of strength by making the pier more or less hollow. However, when this is done the remaining concrete should be well reinforced. It is not advisable in all cases to dispense with any of the filling, for massiveness or weight tends to reduce vibration.

The hollow pier is a compromise between the solid and the cylinder pier; it is less expensive than the former but has somewhat less stability and rigidity; it is more expensive than the latter but is far more stable and makes a more attractive substructure.

The river piers of the Municipal bridge across the Mississippi River at St. Louis, Mo., illustrate the hollow type of pier. As

shown in Figs. 14-7*a* and *b*, the part above high water consists of a tall battered shaft with a large hollow interior space, virtually forming two independent shafts braced together with a well-reinforced arch at the top and walls of masonry on the sides, the latter also serving to give it the appearance of a solid pier. There is a hollow space of less size below high water. This pier is also of interest on account of the shape of cutwater, which, as shown in the plan view (Fig. 14-7*a*), is a combination of the straight and

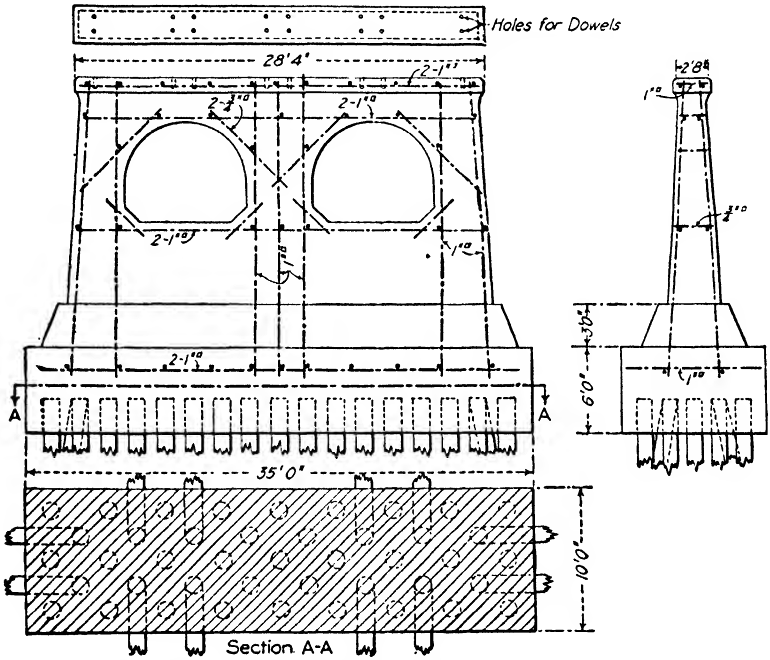


FIG. 14-7*c*.—Piers for Bush and Gunpowder River Bridges.

curved types for the upstream end and semicircular for the downstream end.

A hollow pier resting on a pile foundation and supporting reinforced-concrete slabs is illustrated in Fig. 14-7*c*. The concrete for the footings was a 1:2:4 mixture, while that for the pier shaft was a 1:$2\frac{1}{2}$:5 mixture. In all, 186 piers of this type were used on two bridges of the Pennsylvania Railroad.

The piers of the Sparkman Street bridge, Nashville, Tenn., are shown in Fig. 14-7*d*.

They consist of two concrete towers extending from bridge seat to footing course, and battered on all sides $\frac{1}{2}$ in. to 1 ft., being braced together

by a reinforced-concrete arch and corbeled coping course at the top, and by reinforced side curtain walls from the footing course up to the high-water line. The curtain walls are 2 ft. thick at the top, carried down plumb on the inside, and battering with the towers on the outside. The walls are reinforced with a heavy meshed fabric placed near both inside and outside

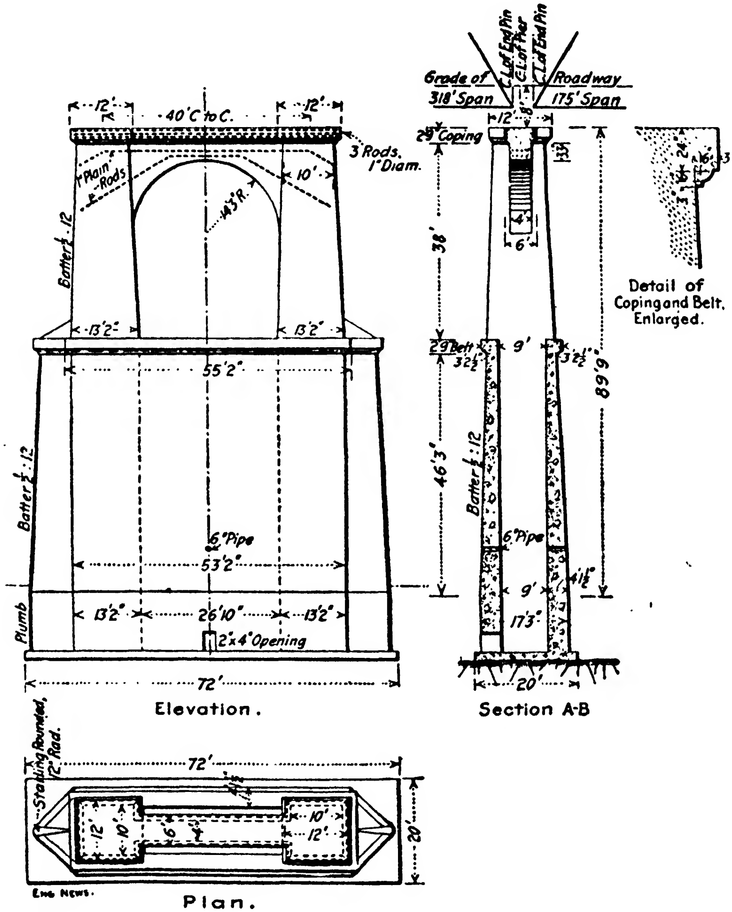


FIG. 14-7*d*.—Typical Channel Pier, Sparkman Street Bridge, Nashville, Tenn. Arched Above and Hollow Below Starling-coping Course.

faces, this fabric extending also entirely around the towers up to the top of the curtain walls.[1]

14-8. Timber Piers. Two general types of timber piers are used, the pile and the crib, or a combination of the two. Pile piers are composed of two parts, the supporting pier and the nose. These

[1] *Eng. News*, vol. 26, p. 576, Nov. 25, 1909.

two parts are sometimes independent structures, but more often they are combined to form a single unit. The main pier is composed of two pile bents spaced from 4 to 8 ft. apart, the number of piles and the amount and position of sway and longitudinal bracing depending on the penetration of the piles, the height of pier, superimposed load, and ice and stream-flow conditions.

The type of timber pier, used by the Rock Island Lines for a bridge over the Platte River, is shown in Fig. 14-8*a*. This pier is composed of two rows of six creosoted piles, capped with concrete. The pier is sheathed on the outside and stiffened with diagonal

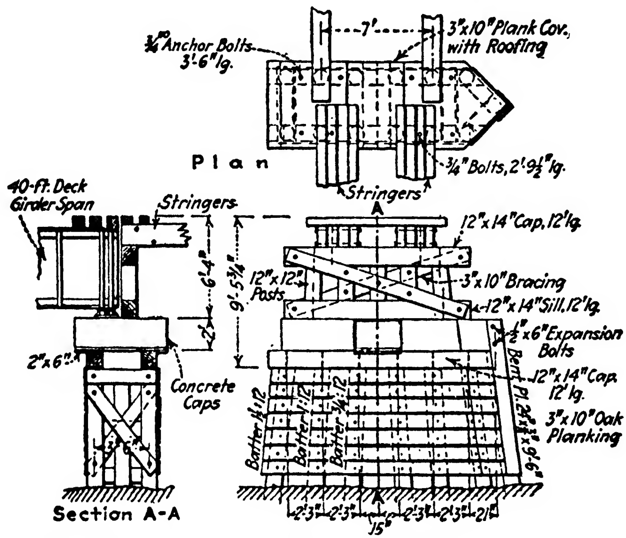


FIG. 14-8*a*.—Timber Pile Pier for Bridge over Platte River.

bracing. A batter pile at the nose serves as an ice breaker, the nose being protected by a bent plate spiked to the sheathing. The 24-in. concrete slab is reinforced with a network of bars near the top and bottom faces and with a stirrup system. This slab serves the double purpose of distributing the load and protecting the pier against fire.

Figure 14-8*b* illustrates the type of pier used in the construction of the Alaska Government Railroad. Here, four rows of piling are used with diagonal cross and longitudinal bracing. The outside sheathing adds stiffness to the structure and prevents lodging of ice and drift against the pier.

Crib piers, which are used where piles cannot be driven, are built of logs or of square timbers. The main body of the pier is

usually rectangular and is divided into compartments by crossties. The logs or timbers may be halved at the corners to make a tight crib or laid one on the other without framing, thus forming a crib with openings. Where round logs are used, they should always be flattened at the ends to a bearing surface at least 4 in. wide. Each log or timber should be drift-bolted to the one below with $\frac{5}{8}$-in. bolts

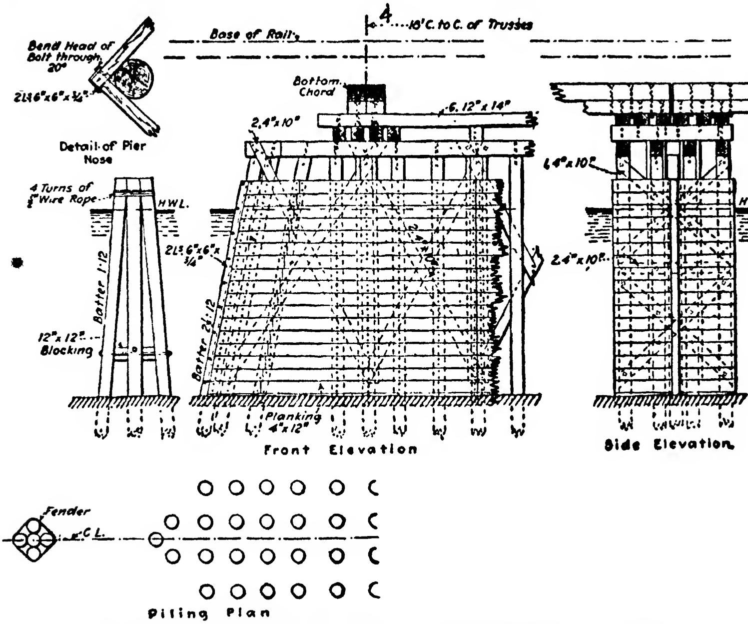


FIG. 14-8*b*.—Timber Pile Pier Used on Alaska Government Railroad.

long enough to extend through the log below and about 2 in. into the second one below.

In streams having a considerable velocity or carrying ice and debris, there should be a wedge-shaped nose and, if there is danger of scouring action, the same kind of tail. The nose angle should be about 60 deg., and it is advisable to give the nose a slope.

If the pier is to be sunk into a soft foundation, the crossties should be far enough above the bottom of the crib to permit easy sinking, and a floor should be placed in the plane of these ties. Cribs are filled with stone to give stability.

14-9. Stability of Piers. The vertical forces to be sustained on any horizontal plane of a bridge pier are the live load, impact load, weight of superstructure, and weight of pier above the plane in question. Impact loads are usually ignored, but more generally on highway than on railroad bridge piers. For the latter, some con-

sideration should be given to impact forces for low piers and for the upper part of high piers.

The lateral forces to be resisted by a railroad pier are tractive forces, wind on train, wind on trusses, wind on pier, river current, and ice pressure. It is customary to specify a tractive force equal to two-tenths of the live load; where the bridge is a double-track structure, some authorities specify a full live load on both tracks and others on one track, the latter being more general. Tests on the Pennsylvania Railroad with electropneumatic brakes indicate a tractive coefficient as high as 0.30. For highway bridge piers tractive forces may usually be neglected.

The wind load on train and trusses should be the same as those used in designing the superstructure, which is customarily taken at 30 lb. per sq. ft. of exposed vertical surface of both trusses and train; or 150 lb. per lin. ft. of bridge for each lateral system, applied at the panel points, and 300 lb. per lin. ft. of train applied at a point 7 ft. above the base of rail. Wind on the end of the pier may be taken at 30 lb. per sq. ft. where the ends are without starlings and 20 lb. per sq. ft. of vertical projection where starlings are present.

The law governing the pressure on bridge piers due to a river current is not definitely known. The formula $P = (Kwv^2)/2g$ is frequently used, in which P is the pressure in pounds per square foot of vertical projection, K a constant, v the velocity of current in feet per second, w the weight of a cubic foot of water, and g the acceleration due to gravity (approximately 32.2 ft. per sec. per sec.). A value often used for $(Kw)/2g$ is 1.5 for flat surfaces and one-half of this for rounded surfaces, with a minimum of 150 lb. per sq. ft. for flat surfaces subjected to freshets and 50 lb. in tidal streams, with one-half of these values for rounded ends.

Experiments show that the velocity varies with the depth approximately as the ordinates of an ellipse, the maximum being somewhat below the surface. The center of pressure is commonly assumed at one-third the distance from the water surface to the river bed. This assumption is on the safe side.

Ice exerts its greatest pressure when in the form of a field of moving ice forcing its way past the pier. In this condition the ice is more or less soft. A value of 50,000 lb. per ft. of pier width for a 10-in. thickness of ice (417 lb. per sq. in.) is often used for flat surfaces, and one-half of this value for rounded surfaces. Other thicknesses will have proportionate values. For the North Side Point bridge, Pittsburgh, Pa., the river piers, which had rounded ends, were designed to resist a horizontal ice pressure of 48,000 lb.

per lin. ft. of width. A value used in the design of a number of large dams in this country is 47,000 lb. per lin. ft. of width.

Uplift is considered by many engineers in designing piers. As a result of a questionnaire by the American Railway Engineering Association in 1917 it was found that approximately two-thirds of the railroads design for uplift.

To be stable, a pier must be safe against sliding and crushing and should be free from tension on any horizontal plane and on the base. The unit sliding force is found by dividing the resultant horizontal force above the section by the area of the section.

The maximum compressive stress is given by the formula

$$f = \frac{P}{A} + \frac{Mc}{I} + \frac{M'c'}{I'},$$

where P denotes the total vertical load; A the area of the section; M and M' the moments due to forces at right angles to and parallel to the long axis of the pier, respectively; c and c' one-half the width and length of pier, respectively; and I and I' the moments of inertia of the pier section about the long and short axis, respectively. The minimum stress is found by making the last two terms of this equation negative. In designing a pier, if on analysis the minimum stress is negative, the section should be increased until a positive value is obtained.

If the section cannot take tension, the above formula does not apply where the minimum stress becomes negative. Such a case is best handled by the use of diagrams, as illustrated by Fig. 14-9*a*, which is applicable for the solution of rectangular footings. The maximum stress is given by the formula

$$f = \frac{KP}{A}.$$

The longitudinal eccentricity is M'/P and the transverse M/P. With these known, the value of K is easily taken from the diagram. For example, with a longitudinal eccentricity of 0.3 and a transverse eccentricity of 0.4, the value of K is approximately 19.

For formulas for stresses on rectangular footings due to doubly eccentric loads, the reader is referred to a valuable article by M. G. Findley in the *Engineering News-Record*, vol. 85, p. 494, Sept. 9, 1920.

The forces resisting sliding are friction of masonry on masonry for stonemasonry piers, the shearing strength of the concrete for concrete piers, and a combination of both for combination piers.

For a table giving friction values for various kinds of stonemasonry and for the shearing strength of concrete, see "American Civil Engineers' Pocket Book." If the pier dimensions at the top accord with standard practice as outlined in Art. 14-3 and if the pier has the conventional batter of 1 in 12 or 1 in 24, all sections will be amply safe against sliding.

Douglas[1] recommends the following allowable compressive unit stresses in pounds per square inch: stonemasonry with 1:2 portland

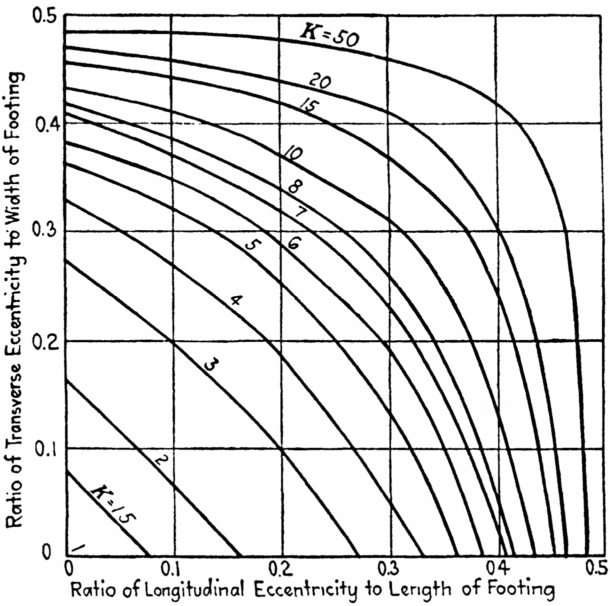


FIG. 14-9a.—Diagram of Coefficient for Eccentric Loads.

cement mortar and joints not over $\frac{1}{2}$ in. thick, granite, 700; hard limestone, 600; medium limestone and marble, 500; soft limestone and sandstone, 400; where joints are over $\frac{1}{2}$ in. thick, 450 lb. for all kinds of sound building stones; for 1:2:4 concrete, 450; 1:3:6 concrete, 350; and 1:4:8 concrete, 250.

14-10. Example of Pier Design. The following example, which analyzes the pressures on the foundation of pier 5 (Fig. 14-10a) of the Tennessee River bridge of the Illinois Central Railroad, is taken in part from an article by W. M. Torrance in *Engineering News*, vol. 53, p. 548. Wind on pier, current, and ice were not considered in the original article. The bed of the river is slightly exposed at low water.

[1] See "American Civil Engineers' Pocket Book," 5th ed., p. 888.

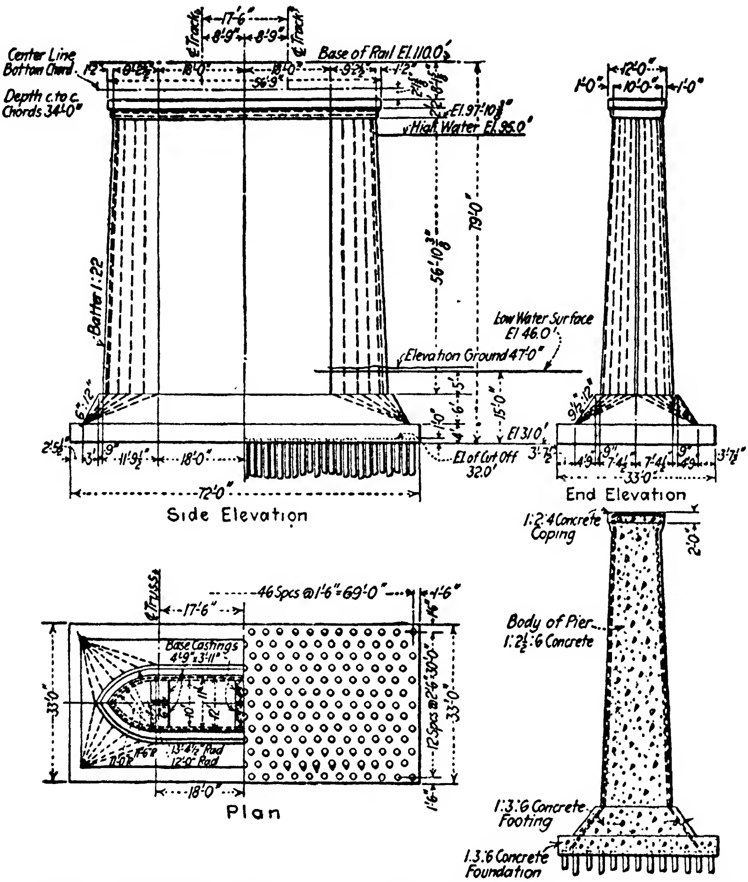


FIG. 14-10a.—River Pier 5, Illinois Central Railroad Bridge over Tennessee River, Gilbertsville, Ky.

Yardage of concrete:

Upper 2 ft. of coping, 1:2:4 concrete; area in plan, 609.3 sq. ft.; volume	45.1 cu. yd.
Lower 2 ft. of coping, 1:2½:6 concrete; area in plan, 548.5 sq. ft.; volume	40.6 cu. yd.
Shaft of pier, 1:2½:6 concrete; top area, 489.4 sq. ft.; bottom area, 782.8 sq. ft.; medium area, 638.7 sq. ft.; volume by prismoidal formula (for height 56.86 ft.)	1,343.2 cu. yd.
Footing course, 1:3:6 concrete; top area, 883.8 sq. ft.; bottom area, 1,738 sq. ft.; volume by end areas	291.3 cu. yd.
Foundation course, 1:3:6 concrete; area in plan, 2,376 sq. ft.; volume	352 cu. yd.

Summary:

1:2:4 concrete in coping...........	45.1 cu. yd.	
$1:2\frac{1}{2}:6$ concrete in coping and shaft.	1,383.8 cu. yd.	
1:3:6 concrete in footing and foundation........................	643.3 cu. yd.	
Total...		2,072.2 cu. yd.
Weight of pier at 155 lb. per cu. ft., 2,072.2 × 27 × 155		8,672,000 lb
Dead load, three trusses with ballast floor, 9,162 × 300		2,738,000 lb.
Live load from 300 ft. of double-track train loads, 5,000 × 2 × 300..................................		3,000,000 lb.
Total gravity load on foundation...................		14,410,000 lb.

Using a tractive coefficient of 0.20, the

Tractive force = 3,000,000 × 0.2 = 600,000 lb.

Assuming this tractive force to act along the center line of the lower chord the

Tractive moment = 600,000 × 73 = 43,800,000 ft.-lb.

To get the moment of forces transverse to the bridge:

Moment of wind on upper lateral system, 150 × 300 × 107..................................	4,815,000 ft.-lb.
Moment of wind on lower lateral system, 150 × 300 × 73..................................	3,285,000 ft.-lb.
Moment of wind on train, 300 × 300 × 86........	7,740,000 ft.-lb.
Total moment from wind on superstructure	15,840,000 ft.-lb.

The projection, on a vertical plane transverse to the long axis of the pier, of the part of the pier subjected to wind at low water is 724 sq. ft. and the distance from the foundation bed to the center of gravity of this area is 43.9 ft.

Moment due to wind on pier at low water,
20 × 724 × 43.9 = 635,700 ft.-lb.
Moment due to river current,
$0.75 \times 10^2 \times 588 \times 48 = 2,116,800$ ft.-lb.
Moment due to ice pressure,
25,000 × 10.25 × 63.5 = 16,271,800 ft.-lb.

The following computations of unit loads on base are made by (*a*) assuming the earth to take all the load and (*b*) assuming the piles to take it:

Direct load on base due to

Weight of Superstructure

Per square foot........	2,738,000/(72 × 33) =	1,152 lb. =	0.58 ton
Per pile...............	2,738,000/306 =	8,950 lb. =	4.47 tons

Weight of Substructure

Per square foot........	8,672,000/(72 × 33) =	3,650 lb. =	1.82 tons
Per pile...............	8,672,000/306 =	28,340 lb. =	14.17 tons

Live Load

Per square foot........	3,000,000/(72 × 33) =	1,264 lb. =	0.63 ton
Per pile...............	3,000,000/306 =	9,800 lb. =	4.90 tons

Uplift at High Water

Area of pier at high water, 504.6 sq. ft.; volume of shaft of pier above high water, 52.7 cu. yd.; total uplift, 1,933.8 × 62.5 × 27 = 3,263,000 lb.

Per square foot........	3,263,000/(72 × 33) =	1,373 lb. =	0.69 ton
Per pile...............	3,263,000/306 =	10,660 lb. =	5.33 tons

Uplift at Low Water

Area of pier at low water, 755.6 sq. ft.; volume of pier shaft below low water, 143.5 cu. yd.; total uplift, 786.8 × 62.5 × 27 = 1,327,700 lb.

Per square foot........	1,327,700/(72 × 33) =	559 lb. =	0.28 ton
Per pile...............	1,327,700/306 =	4,340 lb. =	2.17 tons

The moment of inertia of the base in biquadratic feet about an axis through the center of gravity and parallel with the long axis of the pier is $(72 \times 33^3)/12 = 215{,}600$.

The maximum and minimum pressures on the base due to tractive force are

$$\frac{43{,}800{,}000 \times 16.5}{215{,}600} = \pm 3{,}350 \text{ lb. per sq. ft.} = \pm 1.67 \text{ tons per sq. ft.}$$

The moment of inertia of the pile tops about an axis through the center of gravity and parallel with the long axis of the pier and in units of the area of one pile top times quadratic feet is $2[24(5^2 + 10^2 + 15^2) + 23 \cdot (2.5^2 + 7.5^2 + 12.5^2)] = 26{,}860$.

The maximum and minimum loads per pile due to tractive force are $(43{,}800{,}000 \times 15)/26{,}860 = \pm 24{,}450$ lb. $= \pm 12.22$ tons.

The moment of inertia of the base in biquadratic feet about an axis through the center of gravity and parallel with the short axis of the pier is $(33 \times \overline{72}^3)/12 = 1{,}026{,}000$.

The maximum and minimum pressures per square foot on the base due to the following:

For wind on trusses,

$$\frac{8{,}100{,}000 \times 36}{1{,}026{,}000} = \pm 284 \text{ lb.} = \pm 0.14 \text{ ton.}$$

For wind on train,

$$\frac{7{,}740{,}000 \times 36}{1{,}026{,}000} = \pm 272 \text{ lb.} = \pm 0.14 \text{ ton.}$$

For wind on pier,

$$\frac{635{,}700 \times 36}{1{,}026{,}000} = \pm 22.3 \text{ lb.} = \pm 0.01 \text{ ton.}$$

For river current and ice,

$$\frac{18{,}389{,}000 \times 36}{1{,}026{,}000} = \pm 645 \text{ lb.} = \pm 0.31 \text{ ton.}$$

The moment of inertia of the pile tops about an axis through the center of gravity and parallel with the short axis of the pier, and in units of the area of one pile top times quadratic feet (neglecting moment of inertia about the gravity axis of the individual pile tops), is

$$2[7(1.5^2 + 4.5^2 + \cdots 34.5^2) + 6(3^2 + 6^2 + 33^2.)] = 127{,}100.$$

The maximum and minimum loads per pile are as follows:
For wind on trusses,

$$\frac{8{,}100{,}000 \times 34.5}{127{,}100} = \pm 2{,}189 \text{ lb.} = \pm 1.10 \text{ tons.}$$

For wind on train,

$$\frac{7{,}740{,}000 \times 34.5}{127{,}100} = \pm 2{,}100 \text{ lb.} = \pm 1.05 \text{ tons.}$$

For wind on pier,

$$\frac{635{,}700 \times 34.5}{127{,}100} = \pm 173 \text{ lb.} = \pm 0.087 \text{ ton.}$$

For river current and ice,

$$\frac{18{,}389{,}000 \times 34.5}{127{,}100} = \pm 4{,}991 \text{ lb.} = \pm 2.50 \text{ tons.}$$

It will be seen that the maximum pressure, assuming no uplift, is 5.29 tons per sq. ft., or 40.41 tons per pile, while with uplift the values are, respectively, 4.71 and 35.83. The minimum values show that compression always exists, although at some points it is very slight.

Regarding the effect of uplift, in a case like this, where water is more or less free to get under the pier, there is no question of its action. On the other hand, it cannot act with full hydrostatic pressure on account of the presence of gravel and of the pile tops bearing against the pier.

In this pier, where the top is but a slight distance above high water, wind on pier cannot act simultaneously with ice and current, or, at least, that which acts may be neglected. In computing the minimum pressure, the live load is included as the negative values due to tractive force and wind on train overbalanced the positive value due to direct pressure. In finding the maximum pressure by

considering uplift, the conditions obtaining at low water were used, since these give a greater value than for high water. In getting the minimum values with uplift, high water was used.

Summary of Unit-loading on Foundation

		Tons per square feet	Tons per pile
Weight of superstructure		0.58	4.47
Weight of pier		1.82	14.17
Live load		0.63	4.90
Uplift at high water		0.69	5.33
Uplift at low water		0.28	2.17
Tractive force		1.67	12.22
Wind on trusses		0.14	1.10
Wind on train		0.14	1.05
Wind on pier		0.01	0.09
River current and ice		0.31	2.50
Assuming no uplift	maximum	5.29	40.41
	minimum	0.77	6.67
Assuming full uplift	maximum	4.71	35.83
	minimum	0.08	1.34

The above maximum stress may be checked by the use of Fig. 14-9*a*. Not considering uplift, the transverse eccentricity is 3.04 ft. and the longitudinal eccentricity 2.35 ft. With the two ratios of 0.092 and 0.33 the value of K is found to be approximately 1.75. The value of P/A for dead and live loads is 3.03 tons per sq. ft.; hence the maximum stress from the formula $f = KP/A$ is 5.30 tons per sq. ft.

For the study of the horizontal section of the pier, the same method is to be followed as in obtaining the pressure on the base, except that uplift will be omitted.

CHAPTER XV

DOUBLE-SHAFT AND PIVOT PIERS

15-1. Double-shaft Piers with Metal Shells. For piers built on land, in lakes, and in tidal streams, the lateral forces due to river current and ice may be very small. In such cases the massive types of piers described in the preceding chapter furnish stability and strength far in excess of the requirements. When vertical loads alone are carried, maximum economy results by using two vertical shafts tied together. The shafts may consist of metal cylinders filled with concrete or they may be of reinforced concrete.

The metal shell may be cast iron, wrought iron, or steel, and the pier may be founded on piles or on bedrock, or they may be a continuation of open-cylinder caissons (Art. 9-5) or of pneumatic caissons (Art. 10-8).

Where piles are used and the top stratum consists of silt or other soft soil, this silt should first be excavated to firm material in order that the piles may have good lateral support. Care should also be taken to carry the excavation to below low-water level, as well as to a depth free from danger of scour. Where the cylinder is of small diameter, the piles are driven prior to placing the metal shell. For larger cylinders the shells are often placed prior to driving the piles. In either case the shells are usually driven a few feet into the soil. If clay is penetrated, it is sometimes possible to pump out the water and place the concrete filling in the dry; otherwise a few feet of concrete is placed through the water, usually by the tremie method, and allowed to harden, after which the cylinder is pumped out and the remainder of the concrete placed in the dry.

Where the bottom is rock or hardpan, it is only necessary to clean and level off the site, to place the cylinder, and to fill it with concrete. Where necessary, holes may be drilled in the rock and steel rails grouted in to develop resistance to horizontal forces.

15-2. Examples of Metal-shell Piers. The Tensas River bridge in Alabama, built in 1870, was one of the early large structures using this type of foundation. The shells, of cast iron $1\frac{1}{8}$ in. thick, had exterior diameters of 4 and 6 ft., and were in sections 10 ft. long, the sections being united by bolts through interior flanges 2 in. thick and

3 in. wide. For the fixed spans of the bridge each pier was composed of two 6-ft.-diameter cylinders 16 ft. apart, while the pivot pier had a central cylinder 6 ft. in diameter and six 4-ft. cylinders arranged hexagonally on the circumference of a circle 25 ft. in diameter.

Squared piles arranged closely together, with 12 in each of the 6-ft. cylinders and five in the 4-ft. ones, were driven to a depth of not less than 20 ft. into the sandy bed of the river. Their tops were tied together with bolts and sawed off at low-water level, 15 ft. above the bed of the river. The cylinders were then sunk 10 ft. into the bed of the river, enveloping the pile clusters, and were pumped out and filled with concrete.

Much larger cylinders than those above described were used in the Norfolk and Western Railway bridge No. 5 across Elizabeth

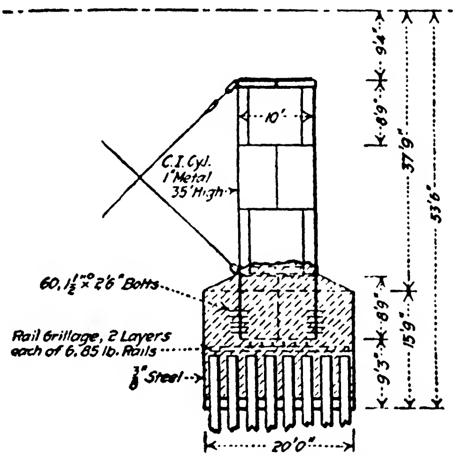


FIG. 15-2a.—Typical Cylinder Pier. Elizabeth River Bridge, Norfolk and Western Railway.

River at Norfolk, Va. The lower part of the cylinder for pier 2 consisted of a ⅜-in. steel shell 20 ft. in diameter and 15 ft. 9 in. long, stiffened by 3½- by 3½-in. angles spaced 5 ft. apart vertically. A temporary upper section of the same diameter and high enough to reach to above water level was attached to act as a cofferdam. The shell was then let down through the water, 23 ft. deep at low tide, and sunk about 18 ft. into the mud by dropping it a few times from a considerable height. The material was then excavated to the bottom edge of the shell, after which 80 piles were driven and cut off by a diver at an elevation 7 ft. above the cylinder bottom. Concrete of a 1:2:4 mixture was deposited through the water to

within 6 in. of the tops of the piles. After allowing this to set 4 or 5 days, the cylinder was pumped out and a 2-ft. layer of concrete, inclosing a grillage of rails, was placed over the tops of the piles to distribute the load more uniformly over them.

As shown in Fig. 15-2*a*, a cast-iron cylinder 10 ft. in diameter was then placed in the larger cylinder. This shell was made in four lengths of 8 ft. 9 in. each and each length was composed of four segments, the whole being bolted together through inside flanges. The metal was 1 in. thick. In the diagram the upper horizontal line represents the base of rail.

Round iron bars $1\frac{1}{2}$ in. in diameter and $2\frac{1}{2}$ ft. long were run through the cast-iron shell near the bottom, and the outside cylinder was then filled with 1:2:4 concrete, which was crowned up on a 30-deg. slope. Concrete was placed in the 10-ft. cylinder to within 2 ft. of the top and a heavy beam grillage was set on this, crowned, and grouted with concrete. The outside cofferdam was removed. Cast iron was used for the upper part of the pier because of its better lasting qualities when only periodically immersed.

In the foregoing examples, some of the piles were extended well up into the cylinder. The advantage of this is the added stability against sliding and overturning. If the cylinders are not subjected to horizontal forces of any considerable magnitude, the piling may be cut at the base of the cylinder or lower. If this is done, the piles are surmounted with a concrete capping or timber grillage and the cylinder placed on it.

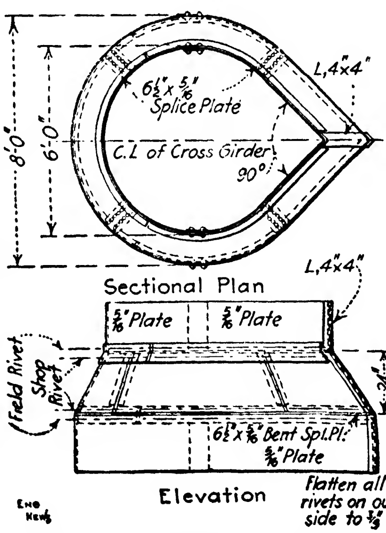


Fig. 15-3*a*.—Cylinder Pier with Pointed End.

15-3. Design and Construction. The size of the cylinder will depend on the load to be supported and the character of the foundation. The area of the base, with the pile foundation, is governed by the number of piles and their spacing; however, if the pier rests on rock or hardpan, the area is governed by the allowable bearing pressure on the same. The area of the upper part of the pier will depend upon the size of the pedestals or base plates of the bridge. In gen-

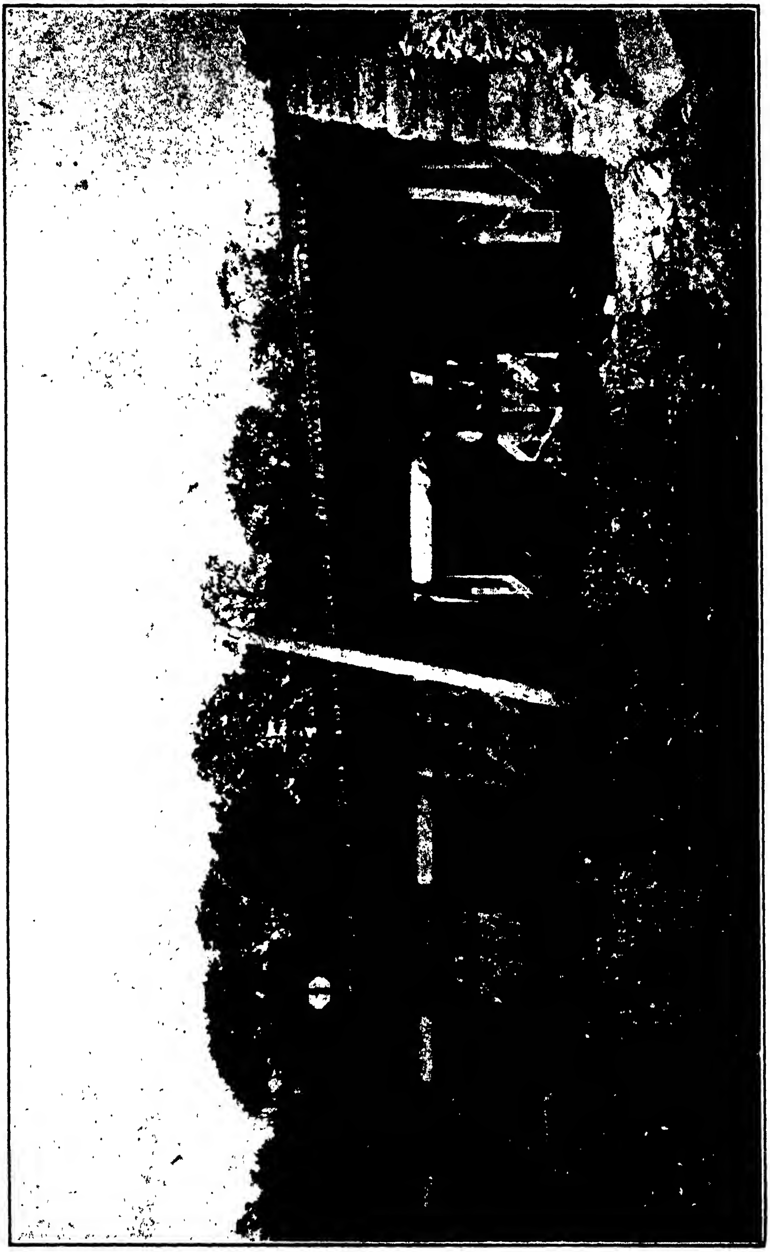


FIG. 15-3*b*.—East Channel Cylinder Piers of Chicago and Northwestern Railway Bridge at Clinton, Iowa.

eral it is advantageous to have the diameter of the cylinder as small as possible, to avoid restricting the waterway and offering resistance to the current, ice, and drift material. Where much ice and drift are present, it may be advisable to use a pointed nose, as illustrated in Figs. 15-3*a* and 9-8*b*.

Where the required diameters at the top and bottom differ materially, a shell having a smaller diameter at the top than at the bottom should be used. This may be done by using two separate shells, as indicated in Fig. 15-2*a*, or by a connection similar to Fig. 15-3*a*, or by using a shell in the form of a frustum of a cone, as illustrated in Fig. 10-8*a*.

The thickness of the shell is usually made just sufficient to take care of the stresses developed in handling and placing. Experience

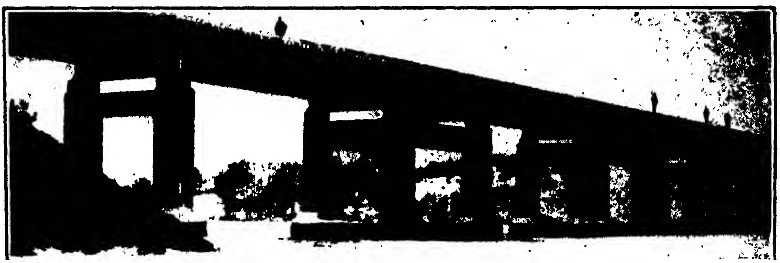


FIG. 15-3*c*.—Oxford Mill Pond Bridge, Chicago and Northwestern Railway.

has demonstrated that it is inadvisable to use less than a $\frac{3}{8}$-in. thickness, although a $\frac{5}{16}$-in. thickness is often specified for highway bridge cylinders. A thickness of more than $\frac{1}{2}$ in. will seldom be required even for large cylinders.

The four possible methods of failure are undermining, settling due to excessive pressure on the foundation, sliding, and overturning. Undermining is mentioned first because of the many failures of highway bridge piers in this country from this cause. Where founded on caissons, there is no danger from this source, but, where founded on piles, care should be taken to have the whole length of the piles well below any possible scouring action, otherwise the foundation may collapse through lack of lateral stability.

To prevent settlement, the foundation, if composed of piles, should be designed in accordance with the rules given in Chap. V with regard to safe loads on piles; or if hardpan or rock, in accordance with safe unit-loads as given in Art. 1-15. The vertical load may usually be assumed as uniformly distributed over the base of the cylinder, for the transverse loads are resisted by a trusslike

action of the cylinders and bracing. Thus, with a two-cylinder pier, in addition to the vertical loads due to the live load, weight of superstructure, and weight of pier, there will be a downward vertical load on one cylinder equal to the moment of the transverse

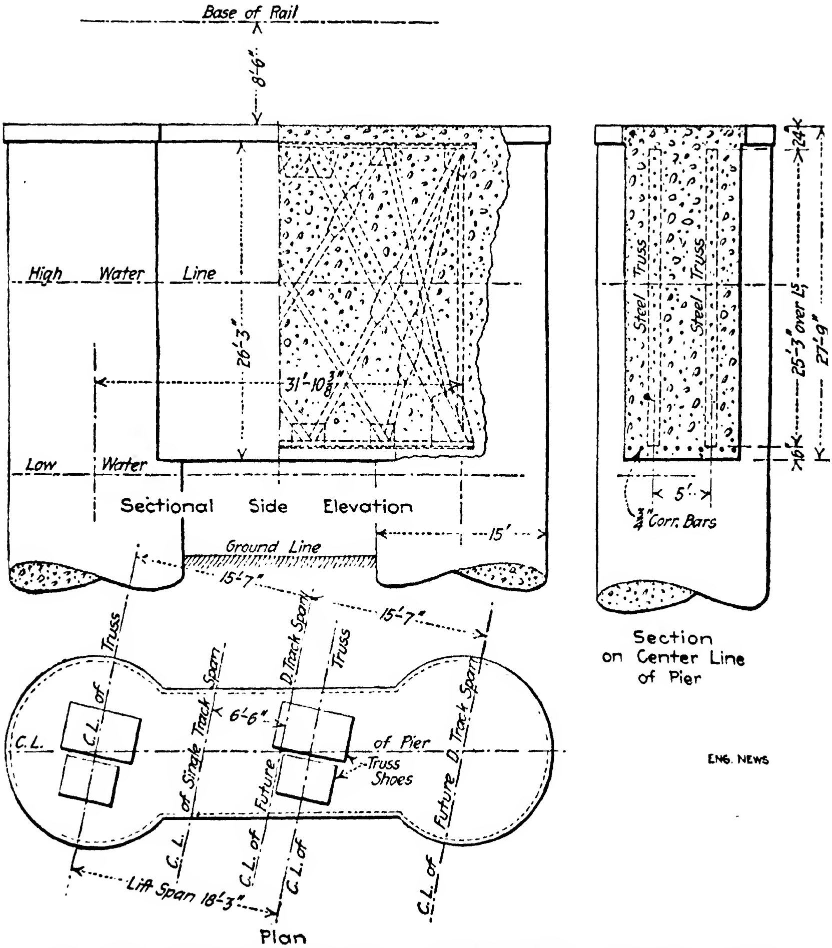


Fig. 15-3*d*.—Cylinder Piers Braced by a Truss Encased in Reinforced Concrete.

loads about the bottom of the pier, divided by the distance between cylinders center to center.

Where forces exist tending to slide the pier, if a pile foundation is used, some of the piles should extend well up into the cylinders; whereas, if the cylinders rest on bedrock, they should be anchored to the rock surface.

To resist overturning, ample bracing should be used between the cylinders. Many forms of bracing are illustrated in the accompanying figures. These include latticed girders, as in Fig. 15-3*b*; plate girders, as in Fig. 15-3*c*; box girder filled with concrete, as in Fig. 10-8*a*; and deep trusses embedded in concrete, as in Fig. 15-3*d*.

15-4. Double-shaft Piers of Reinforced Concrete. General practice now favors the use of reinforced-concrete rather than metal

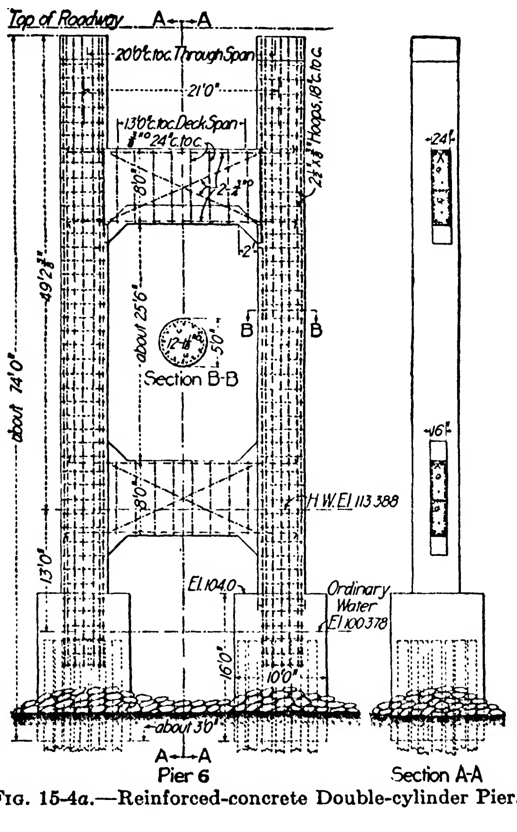


FIG. 15-4*a*.—Reinforced-concrete Double-cylinder Pier.

shells filled with concrete, particularly in the case of high piers. The shafts may be in the shape of cylinders or truncated cones or they may have plane faces with the sectional areas changing either by battering or by the use of steps. The shafts may have a common footing or the footings may be independent. The bracing connecting the shafts may consist of a solid diaphragm of reinforced concrete or it may be in the form of a number of struts placed at intervals.

The use of low and of moderate height piers is illustrated in Fig. 16-6*a*. These piers consist of two battered shafts with independent

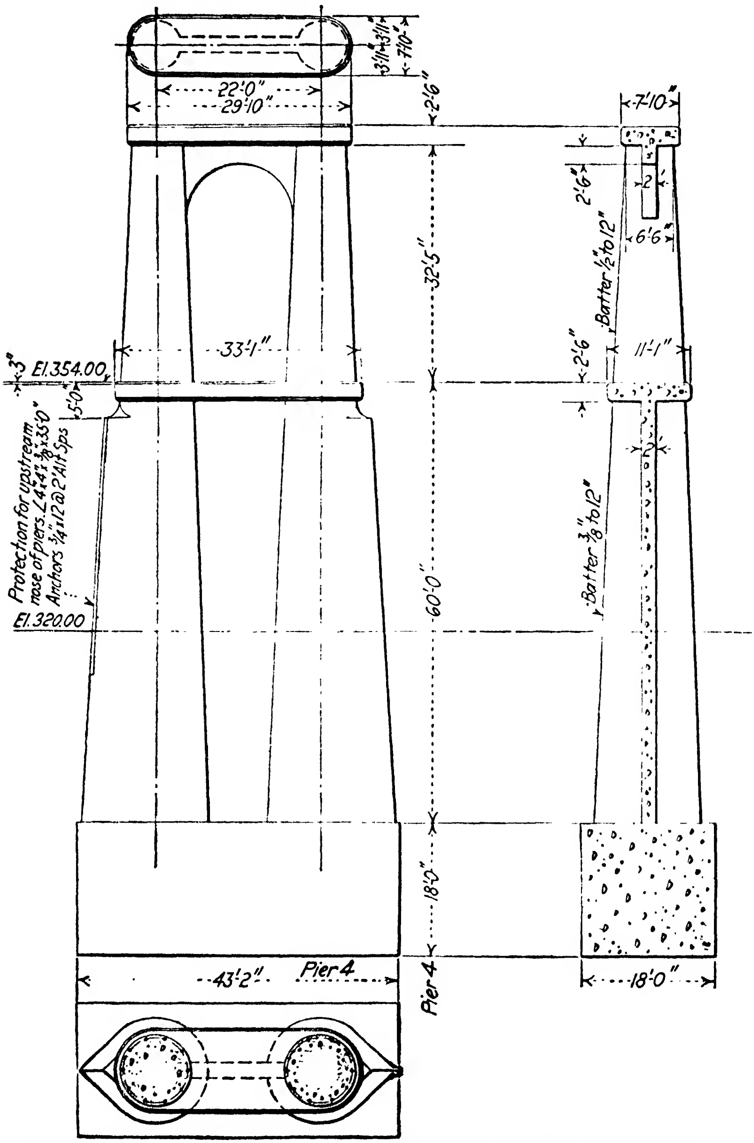


FIG. 15-4*b*.—Pier of the Kennewick-Pasco Bridge over the Columbia River.

footings. The shafts are connected at the top by a reinforced-concrete girder.

FIG. 15-4c.—Kennewick-Pasco Bridge over the Columbia River.

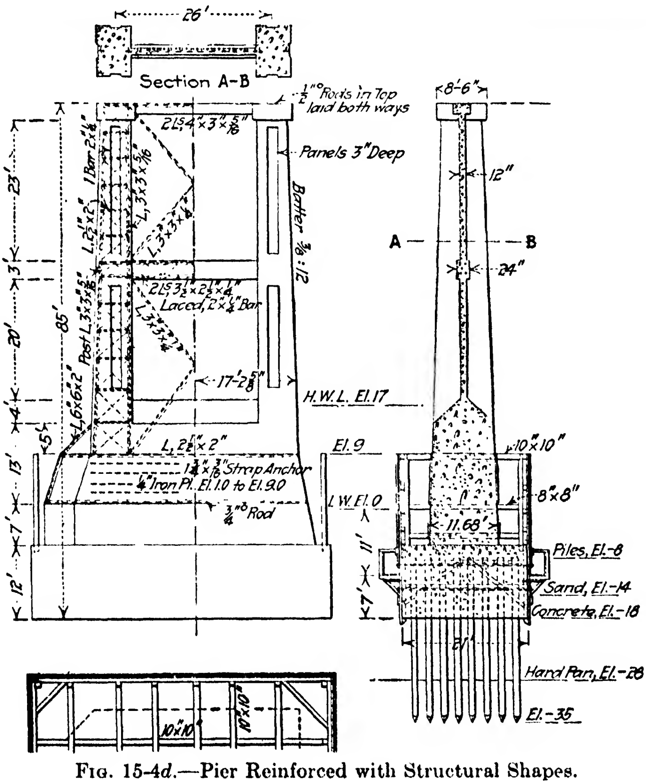


FIG. 15-4d.—Pier Reinforced with Structural Shapes.

Fig. 15-4*a* illustrates the reinforced-concrete cylinder piers of a highway bridge over the St. Croix River at Hudson, Wis. Each pier consists of two cylindrical shafts from 4 to 6 ft. in diameter with independent footings, the shafts being tied together with two diaphragms, the upper one being 2 ft. thick and 8 ft. deep and the lower one 16 in. thick and 8 ft. deep.

The piers of the Kennewick-Pasco bridge over the Columbia River, as shown in Figs. 15-4*b* and *c*, furnish a good example of

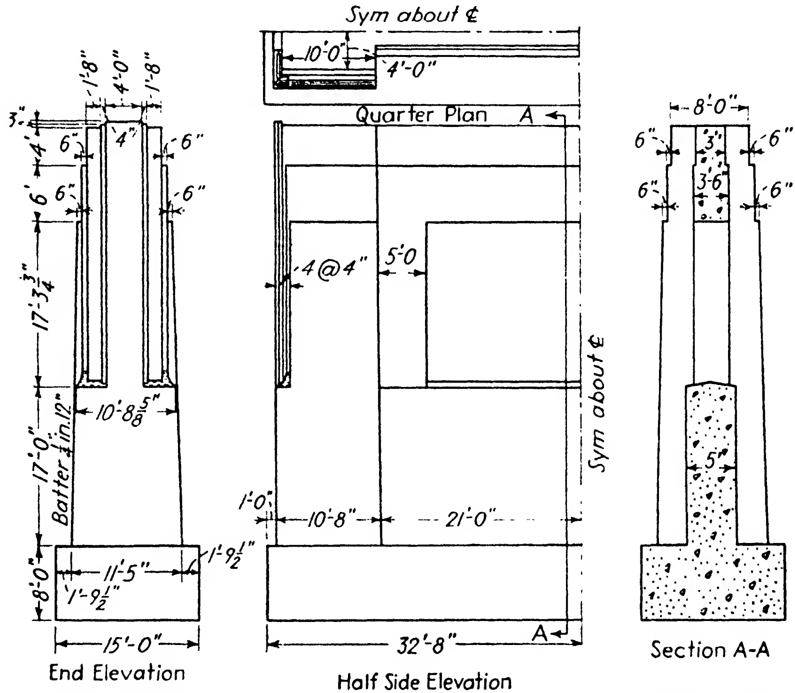


Fig. 15-4*e*.—Reinforced-concrete Double-shaft Pier for Bridge in Charleston, W. Va. (*Courtesy of C. P. Fortney, Consulting Engineer.*)

conical shafts with a common footing. Up to high-water level the shafts are connected with a cross wall of reinforced concrete 2 ft. thick, and the section of the shaft is modified to furnish a cutwater both upstream and downstream, the upstream nose being protected by a 4- by 4- by $\frac{3}{8}$-in. angle 35 ft. long. At the top of the pier the shafts are connected with a coping course, as well as with a cross wall. The coping at high-water elevation serves to break the appearance of extreme height. The shafts are reinforced with $\frac{3}{4}$-in. longitudinal rods and $\frac{1}{2}$-in. hoops.

In place of rod reinforcement, structural shapes are sometimes used for both shafts and diaphragms, as shown in Fig. 15-4*d*. Here each pier consists of a pair of shafts battered $\frac{3}{8}$ in 12 and having a rectangular section. These shafts were paneled on three sides and connected with a 12-in. diaphragm. The structural steel was assembled and placed as a unit.

Figures 15-4*e* and *f* illustrate the 52-ft. piers used on a highway bridge over the Elk River in Charleston, W. Va. The footing consists of a slab of reinforced concrete 15 ft. by 65 ft. 4 in. in plan and 8 ft. high. Two shafts 11 ft. 5 in. wide and 10 ft. 8 in. long at the base rise from this footing, the sides being battered $\frac{1}{4}$ in 12 for a height of 34 ft. $3\frac{3}{4}$ in. Above this elevation two 6-in. steps on each side result in a width at the top of 8 ft. The length at the top is 10 ft. 8 in. for a width of 4 ft. and 10 ft. outside the 4-ft. section. For a distance up to 17 ft. there is a solid concrete diaphragm 5 ft. wide connecting the shafts. For the upper 10 ft. there is another diaphragm 3 ft. thick for the upper 4 ft. and $3\frac{1}{2}$ ft. thick for the next 6 ft. Between the upper and lower diaphragms each shaft is braced by a concrete pedestal 5 ft. wide and $3\frac{1}{2}$ ft. thick. The entire pier is heavily reinforced with rods.

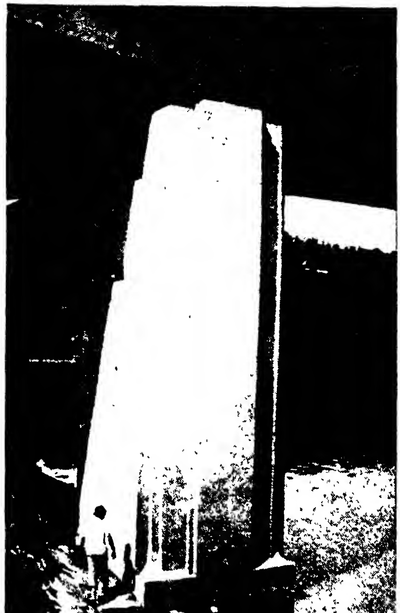


FIG. 15-4*f*.—Double-shaft Pier for Bridge in Charleston, W. Va. (*Courtesy of C. P. Fortney, Consulting Engineer.*)

Figure 15-4*g* shows the double-shaft piers of the Goethals' bridge at Elizabethport, N. J. Here the shafts decrease in section by stepping in the outside face and the two side faces. Deep diaphragms at the top of the piers brace the shafts.

The volume of concrete and weight of reinforcement for piers of the form shown in Fig. 15-4*h* are given in Fig. 15-4*i*.

15-5. Large Cylinder or Pivot Piers. This type of pier, used almost exclusively for the center support of swing spans, resembles the cylinder pier in shape and the ordinary masonry pier in massive-

ness. The same types of foundations, kinds of material and methods of construction are used as for ordinary piers. Where protection against ice and drift is necessary, it is furnished by means of an independent pier, often constructed of long piles.

Figure 15-5*a* illustrates the all-concrete solid pivot pier on piles used for the St. Louis River bridge near New Duluth, Minn. Since

FIG. 15-4*g*.—Goethals' Bridge at Elizabethport, N. J. (*Courtesy of Port of New York Authority.*)

the depth of water was about 28 ft., the pier was constructed in wooden forms inside of a circular steel sheet-pile cofferdam 36 ft. in diameter. After driving the piles and cutting them off about 4 ft. above the dredged bottom, a 6-ft. layer of concrete was placed to form the 36-ft. diameter footing course, the sheet piling serving as a form for the sides. Above this footing course the form for the pier consisted of a wooden-stave water tank 16 ft. high having a side

batter of 1 in. to the foot (Fig. 15-5*b*). The second lift was made by raising the tank and planing a few of the staves to fit the new

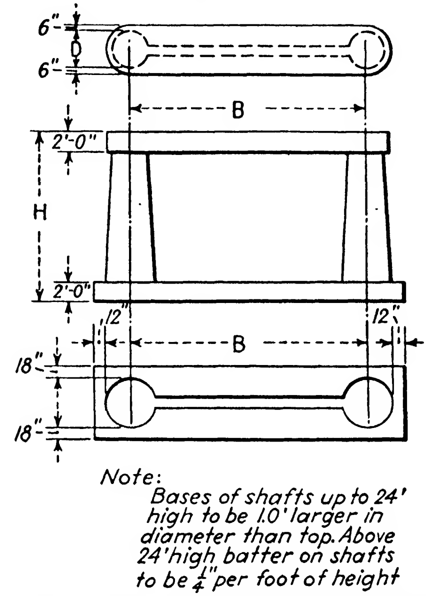


FIG. 15-4*h*.—Reinforced-concrete Double-shaft Pier.

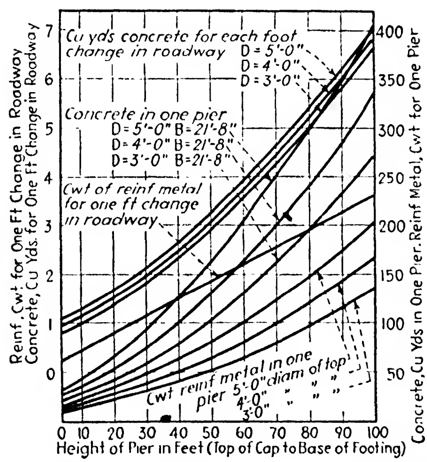


FIG. 15-4*i*.—Quantities in Reinforced-concrete Double-shaft Pier (see Fig. 15-4*h*). (*From Economics of Highway Bridge Types by C. B. McCullough, Gillette Publishing Company.*)

dimensions. For the coping course galvanized iron of the section shown in the illustration was used. A grillage of 24-in. I-beams distributed the load over the pier.

The pivot pier construction at the Dumbarton bridge of the Central California Railroad merits particular attention because of its simple solution of a difficult problem. At the site the depth of water at low tide was 51 ft. and at high tide 58 ft., with a maximum velocity of current of $4\frac{1}{2}$ m.p.h. The bottom consisted of soft mud overlying stiffer material. On account of the great depth of water and velocity of current, the cofferdam process was

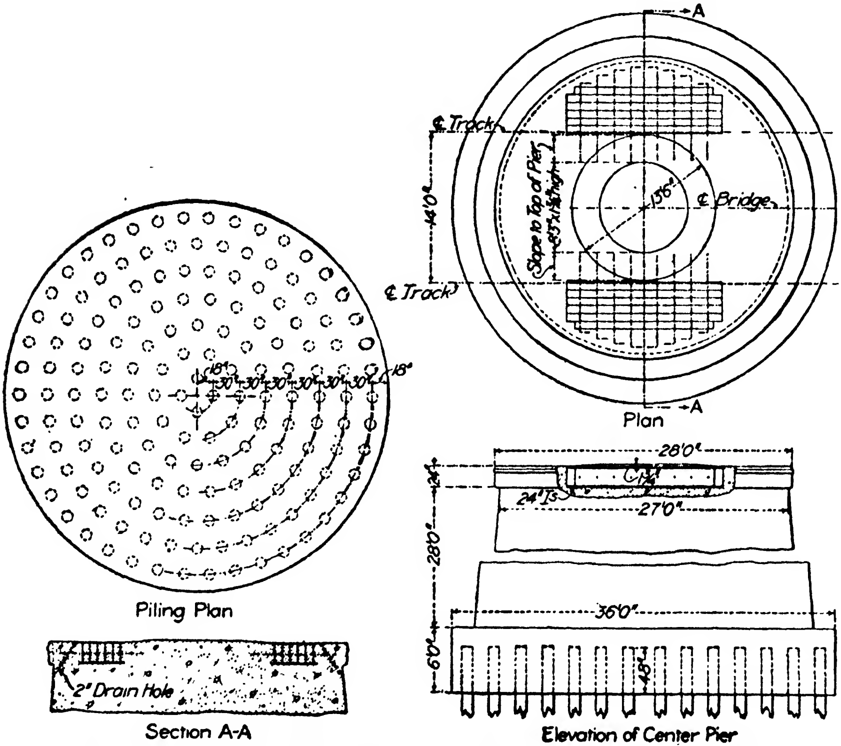


Fig. 15-5a.—Concrete Pivot Pier, St. Louis River Bridge, Duluth, Minn.

not practicable and caisson foundations would have been expensive. Hence it was decided to employ a metal shell with a pile foundation. The cylinder had a diameter of 40 ft. and was 72 ft. 5 in. high in five vertical sections. After dredging out about 10 ft. of the soft material on the bottom, the first section of the cylinder was lowered. This was effected by first lowering a guide frame of structural shapes and driving its feet into the bottom, after which the section of the cylinder was placed around this frame and lowered. Inside this section 141 piles were driven to a penetration of about 30 ft. and cut off below low-water level; some only about 3 ft. below low water and

others near the bottom. On completion of the pile driving, more sections of the shell were added, each section being filled with concrete placed through the water to within 7 ft. of the top before another section was added. This was the highest level at which the top of the concrete would not be disturbed by the tidal current passing over the top.

Where the lateral forces on the piers are small, it is not necessary to extend the piles into the cylinders. In the construction of a pivot

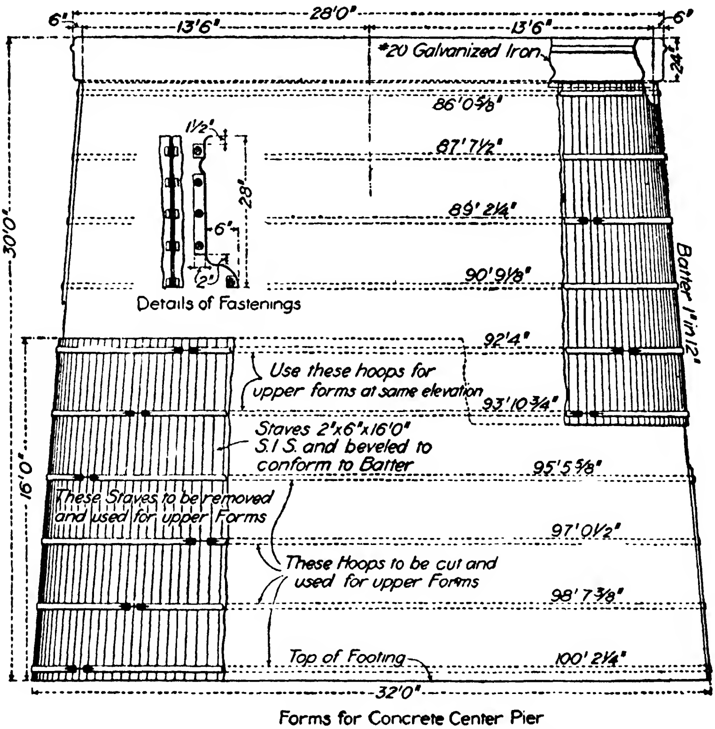


FIG. 15-5b.—Form for Constructing Concrete Pivot Pier.

pier in the Willamette River, Portland, Ore., where the depth of water was 60 ft., piles were driven and cut off near the bottom. A timber grillage extending to within 3 ft. of low-water mark was placed on the piles. On this grillage was placed a steel shell 46 ft. in diameter about 30 ft. high, which was filled with concrete to form the pivot pier.

Figure 9-3*d* shows the pivot pier of the Chelsea Bridge North, Boston, which was faced with stonemasonry. The foundation for this pier is described in Art. 9-3.

As in ordinary bridge piers, the tendency at present is to make the pivot pier of hollow construction, leaving out masonry from that part of the pier that is but slightly stressed.

Figure 15-5c shows the reinforced-concrete hollow pivot pier of the Tennessee River bridge of the Illinois Central Railroad.

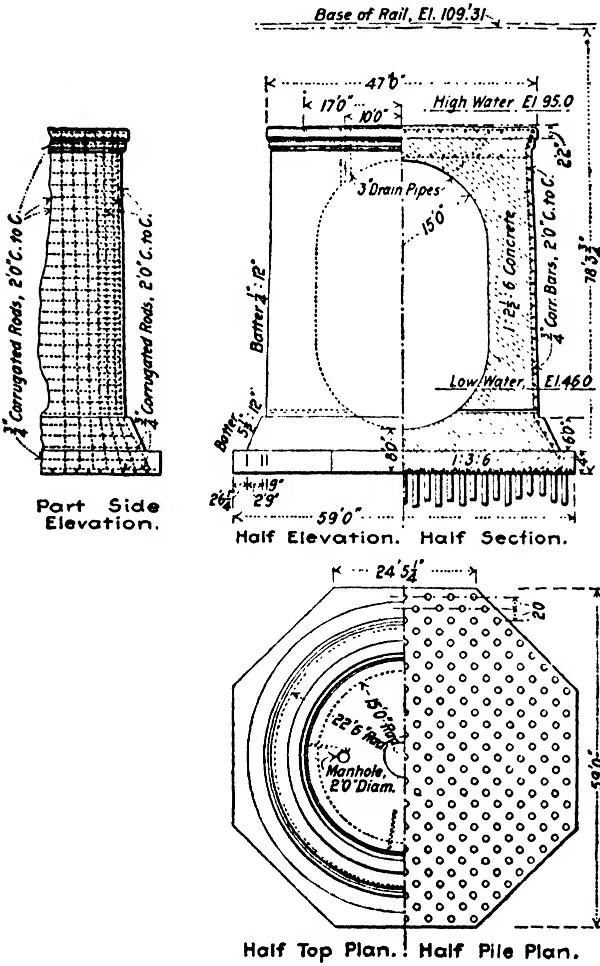


FIG. 15-5c.—Reinforced-concrete Pivot Pier, Illinois Central Railroad Bridge, Gilbertsville, Ky.

The hollow space is domed at the top and bottom. The entire load from the superstructure comes on the pier through a cast-iron track 38 ft. in diameter. The circular center line of the 8-ft. wall has the same diameter. thus avoiding eccentric stresses in the pier.

CHAPTER XVI

BRIDGE ABUTMENTS

16-1. Forms and Dimensions. Bridge abutments are structures at the two ends of a bridge used for the double purpose of transferring the loads from the bridge superstructure to the foundation bed and giving lateral support to the embankment. Abutments are usually of concrete (Fig. 16-1*a*) or of stonemasonry (Fig. 16-1*b*), but they are sometimes formed of steel piling as explained in Art. 7-3.

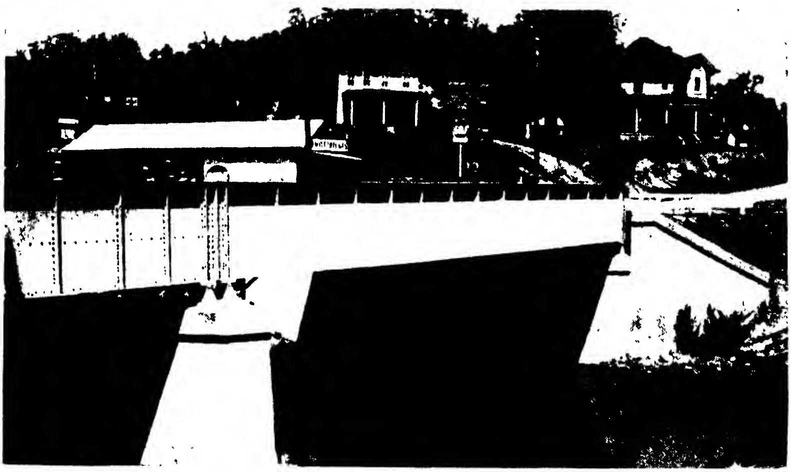


FIG. 16-1*a*.—Bridge with Wing-wall Abutments of Concrete.

The abutment serves both as a pier and as a retaining wall and in the more usual types consists of a breast wall and wing walls. Ordinarily it is independent of the superstructure (Fig. 16-1*a*), but in rigid-frame spans (Fig. 16-1*c*) it is continuous with the superstructure and transmits shears, thrusts, and moments from the same.

In Fig. 16-1*d*, *A* is the bridge seat, which consists of a horizontal surface on which rest the end bearings of the superstructure; *B* is the back or parapet wall, which affords lateral support for the upper part of the embankment; *C* is the main body or stem; and *D* is the footing.

The forces acting on the abutment to hold it in equilibrium are as follows: (*a*) the forces from the superstructure, including vertical dead and live loads, longitudinal traction and braking forces, and transverse wind forces; (*b*) lateral earth pressure against the back of the wall resulting both from the weight of the fill and the live load above; (*c*) the weight of the abutment; and (*d*) the reactions from the foundation bed, both vertical and horizontal.

The three simplest types of abutments are the wing-wall abutment, the U-abutment, and the T-abutment. In the wing-wall type the wings may be at any angle with the breast wall. In grade-

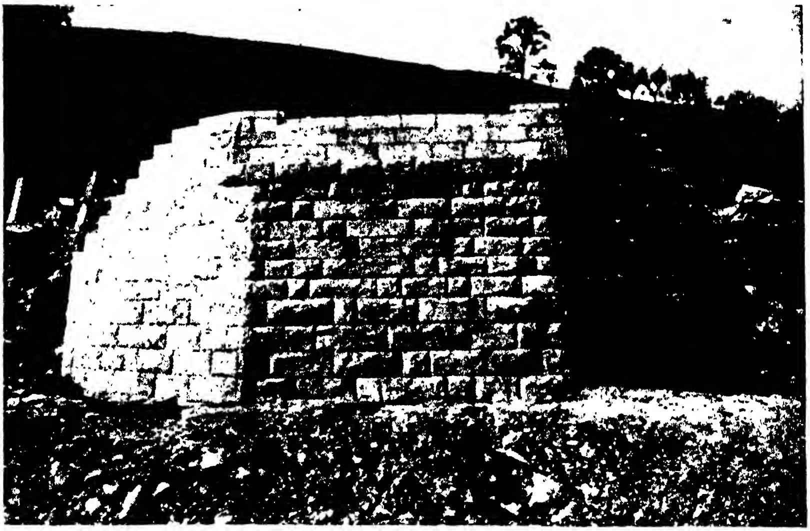


Fig. 16-16.—Wing-wall Abutment of Stone Masonry.

crossing elimination work, the wings are often a continuation of the face wall, while in many locations angles of 30 and 45 deg. are widely used.

The U-abutment is a special form of the wing-wall abutment in which the wings are parallel with the roadway. To keep the embankment slopes from spilling out in front of the face wall when a $1\frac{1}{2}$ on 1 slope is used, for 45-deg. wings the length of the wing wall is approximately equal to the height of the abutment above ground level, whereas for U-abutments and abutments in which the wings are parallel with the front wall, the length is 50 per cent greater.

The T-abutment has the same form of front wall as the other types, but instead of wings it has a solid stem which supports the track or roadway back to a point at which the embankment is at the elevation of the top of the abutment (Art. 16-5).

Abutments may be of the square or skew type, depending on the angle the front wall makes with the roadway. Figure 16-3*a* illustrates the skew type, other illustrations being mostly of the square type.

FIG. 16-1*c*.—Rigid-frame Bridge with Wing-wall Abutment.

The dimensions of a bridge abutment depend on the type and size of the superstructure, height of abutment, character of foundation, and nature of embankment. The same factors govern the dimensions of the front wall in the plane of the bridge seat as for the top of a bridge pier (Art. 14-3) except that bearing areas are required for the end of only one instead of two spans. On the other hand an additional width *e* (Fig. 16-1*d*) is required for the base of the parapet wall, which is customarily made about 0.40 to 0.45 *k* (Fig. 16-1*d*) unless the parapet wall is reinforced.

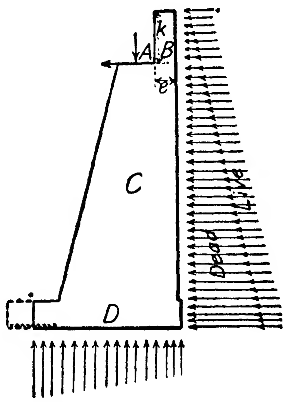


FIG. 16-1*d*.—Section of Abutment Indicating Some of the External Forces Acting upon It.

The width of the bridge seat should not be less than 1 ft. greater than required for the bedplates of steel superstructures, and the length of the bridge seat should not be less than the distance out to out of the bedplates plus 4 ft. Generally the width of the upper surface of the back and slope walls should not be less than 2 ft. for railroad bridges and $1\frac{1}{2}$ ft. for highway bridges. Copings may be pro-

portioned in conformity with the proportions as given for bridge piers in Art. 14-3.

The thickness of the stem may be found by the methods outlined in Art. 16-2. However, owing to the many uncertainties involved in figuring earth pressure, an empirical rule often calls for a thickness at any elevation of not less than 0.4 to 0.45 the height above that elevation.

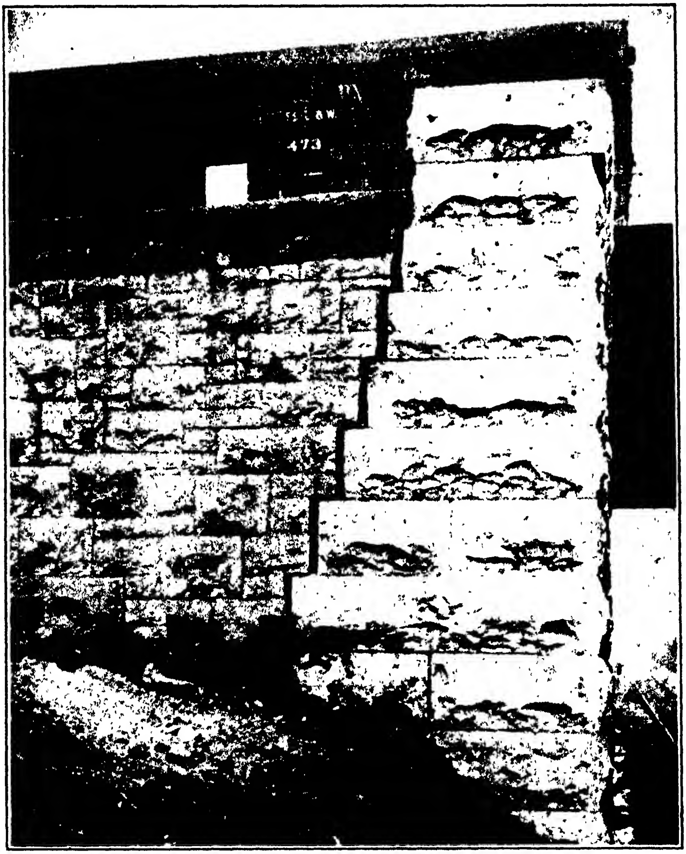


FIG. 16-2a.—Effect of Unequal Bearing on Foundation Bed of Abutment.

16-2. Design and Construction. The vertical loads to be sustained on any horizontal plane of the front wall are the reactions from the weight of the superstructure, live load, and impact load, as well as the weight of the masonry above the plane. Impact is often not included.

The lateral forces parallel to the axis of the bridge are traction and braking forces from the superstructure and earth pressure on the

back of the abutment due to both the weight of the embankment and the live load. At right angles to the axis of the bridge are the wind loads from the superstructure and on the abutment. The latter two are usually neglected, their effect being slight as compared with that of the other forces. Earth-pressure forces are discussed in Arts. 2-6 and 2-11. The magnitude and direction of application of earth pressure are both rather uncertain. As an approximation for determining lateral pressure the earth is often considered as a fluid weighing about 30 lb. per cu. ft.

For stability the solid-gravity abutment must satisfy the same conditions as those given for piers in Art. 14-9. For reinforced-

FIG. 16-3a.—Plate-girder Bridge Showing Use of Skew Abutments. (*Courtesy of F. P. Turner, Principal Assistant Engineer, Norfolk & Western Railway.*)

concrete abutments the base must satisfy these same conditions, while the constituent parts are designed as beams and columns.

Unless the abutment rests on firm, unyielding material, the resultant pressure on the breast wall should strike the base near the center in order to give substantially uniform bearing pressure over the whole base. If the resultant is at the middle third of the base, the nonuniform pressure may cause a differential settlement, resulting in a condition illustrated in Fig. 16-2a in which a crack develops between the front wall and the wing walls. This desired distribution can be obtained by reducing the value of e in Fig. 16-1d, by using a light batter on the back face and a heavy batter on the front face and by extending the footing as shown by the dotted lines. The distance e can be reduced by reinforcing the backwall by the use of vertical rods near the back face.

Concrete abutments should have surface reinforcement as in the case of piers (Art. 14-4). Solid, massive concrete abutments are usually made with a mix approximating 1:2½:5 below the coping and 1:2:4 for the coping. Reinforced-concrete abutments are usually of a mix approximating 1:2:4.

Proper drainage behind the abutment is of primary importance, since earth in a semifluid condition exerts a pressure much greater than ordinary earth pressure. Water collecting behind the abutment and freezing is almost sure to cause cracking. Weep holes through the walls near the ground line, connecting to French drains along the back, make a good drainage system.

The filling back of the abutment should be placed in horizontal layers about 1 ft. thick and thoroughly tamped or rolled before placing the next layer. This will avoid the wedging action which results if earth is dumped from a height.

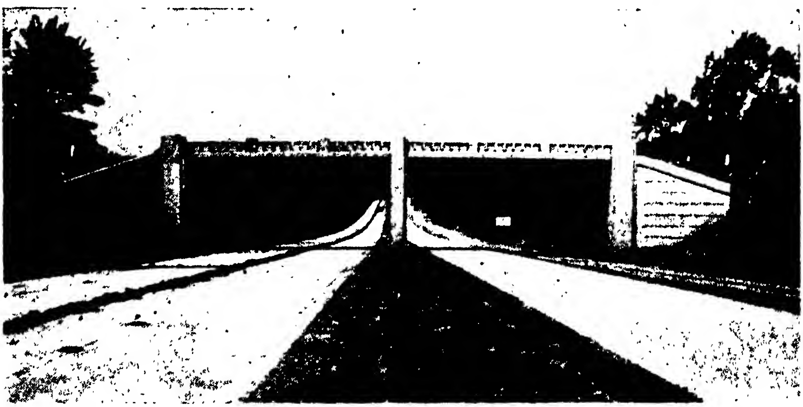


FIG. 16-3*b*.—Showing Wing-wall Abutments with Ornamental Pylons.

16-3. Wing-wall Abutments. This type of abutment may have its wings parallel to, and in line with, the face or breast wall when the bridge is over a highway, but generally the wings are flared back. The wing in the foreground of the left abutment of Fig. 16-3*a* is an example of one parallel with the face wall, while that for the right abutment illustrates a flaring wing. Other examples of flaring wings are shown in Figs. 16-1*a*, 16-1*b*, 16-1*c*, and 16-3*b*. The last-named also illustrates the modern trend in the use of pylons for aesthetic effect.

For stream crossings the wings should always be turned back to protect the embankment against washing out under high water, as

well as to furnish better hydraulic conditions by providing a gradual change in the cross section of the channel.

The required length of a wing wall will be a minimum when the neat line of its face bisects the angle between the face of the breast wall and the shoulder of the fill. According to Baker,[1] for square

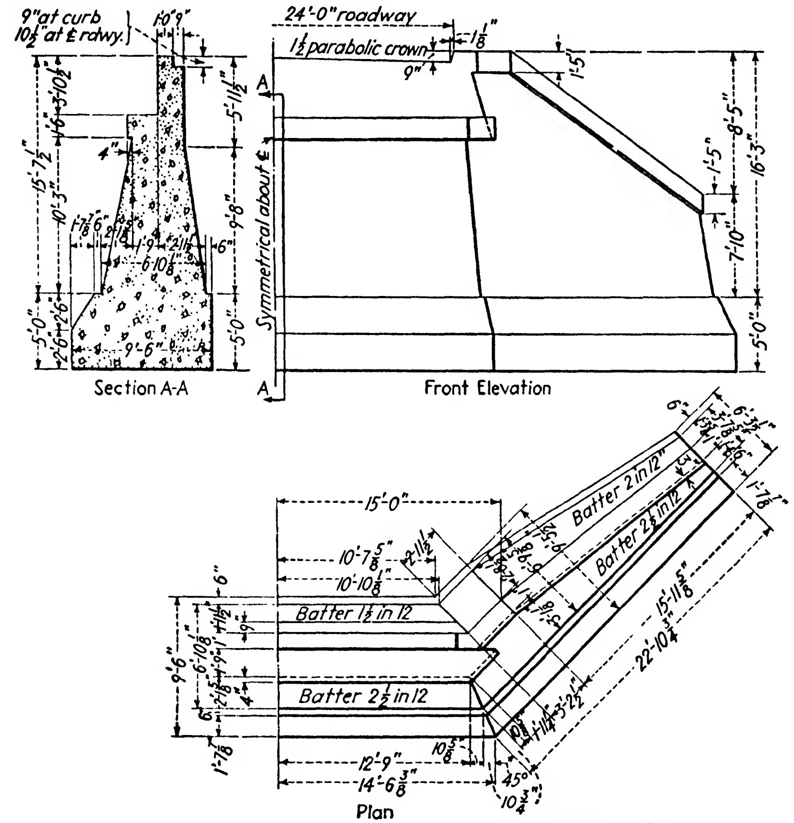
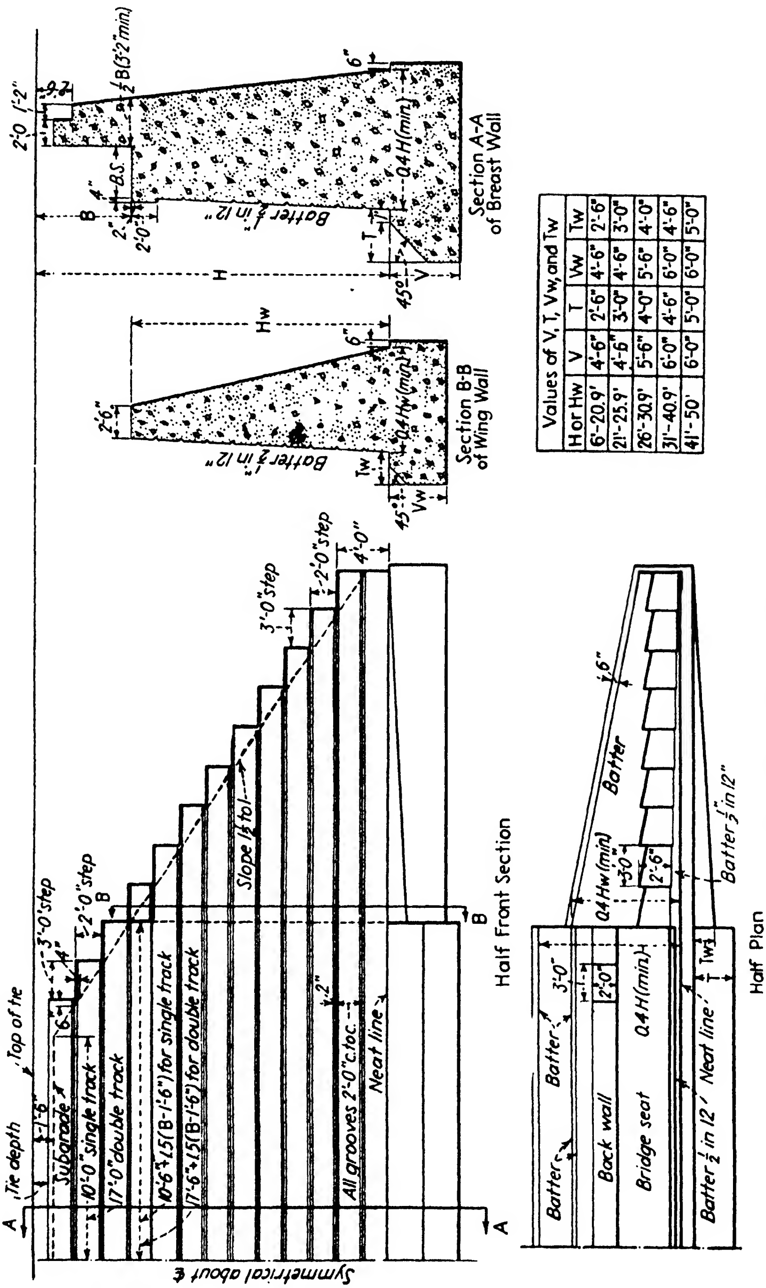


FIG. 16-3c.—Typical Wing-wall Abutment as Used by the State Road Commission of West Virginia.

abutments the proper angle of wing deflection for minimum yardage of masonry is between 25 and 35 deg., if the earth flowing around the toe is to be kept 3 or 4 ft. back of the extension of the neat line of the breast wall.

The wings are designed as retaining walls. The required thickness of the gravity type varies from 0.33 to 0.45 of the height of the wall, a value of 0.4 being widely used.

[1] "Masonry Construction," 10th ed., p. 536, John Wiley & Sons, Inc., New York.



H or Hw	V	T	Vw	Tw
6'-20.9'	4'-6"	2'-6"	4'-6"	2'-6"
21'-25.9'	4'-6"	3'-0"	4'-6"	3'-0"
26'-30.9'	5'-6"	4'-0"	5'-6"	4'-0"
31'-40.9'	6'-0"	4'-6"	6'-0"	4'-6"
41'-50'	6'-0"	5'-0"	6'-0"	5'-0"

Values of V, T, Vw, and Tw

FIG. 16-3d.— Railroad Bridge Abutment.

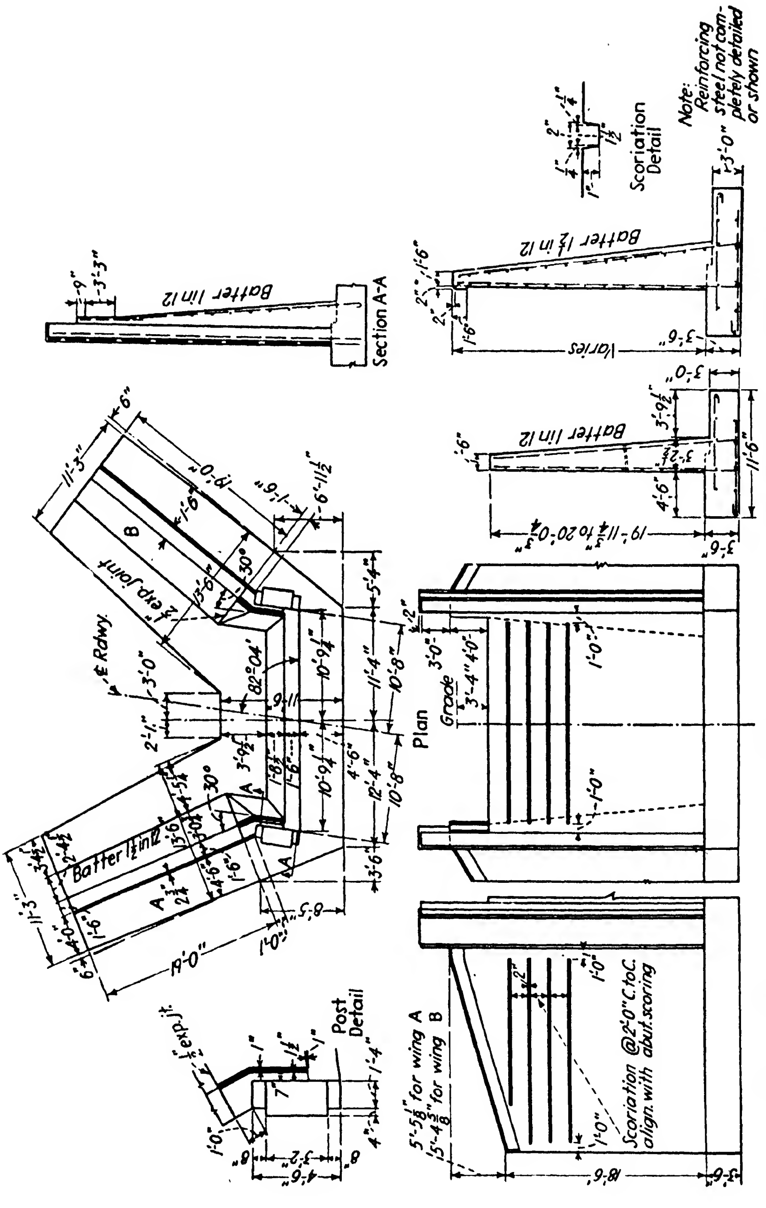


FIG. 16-3e.—Cantilever Type of Abutment. (*Courtesy of R. F. Graef, formerly Chief of Bridges, Pennsylvania Turnpike Commission.*)

Figure 16-3*c* illustrates a typical wing-wall abutment for a highway bridge I-beam span with a 24-ft. roadway, as used by the highway department of West Virginia.

Figure 16-3*d* shows a railroad bridge abutment similar to the type that is standard with the Baltimore and Ohio Railroad Company. Table 16-3*a* gives the yardage of concrete for this type of abutment, these quantities being based on a bridge seat width of 4 ft. Quantities are given for both single- and double-track structures.

TABLE 16-3*a*. YARDAGE IN RAILROAD BRIDGE ABUTMENT. WINGS PARALLEL TO FACE OF ABUTMENT*
(See Fig. 16-3*d*)

Height *H*, feet		10	14	18	22	26	30	34	38	42
Width of fill, 20	*B* = 4	119	175	248	351	503	697	915	1,174	1,467
		13	17	19	27	34	43	52	61	74
	B = 6		175	242	348	508	671	921	1,180	1,456
			18	20	26	35	43	54	64	73
	B = 8		182	254	359	521	690	938	1,182	1,480
			18	22	26	36	52	54	63	74
Width of fill, 34	*B* = 4	169	242	327	459	648	877	1,141	1,435	1,778
		19	23	25	33	42	52	62	72	86
	B = 6		239	321	451	644	846	1,137	1,436	1,758
			23	25	32	43	51	64	74	85
	B = 8		244	333	457	657	860	1,149	1,433	1,777
			24	28	33	44	51	64	73	86

For each value of *B* the upper horizontal row gives the quantity of concrete in cubic yards for one abutment with a depth of footing as given in Fig. 16-3*d*, while the lower horizontal row gives the yardage for each additional foot of depth.

* Taken from data furnished by C. E. Sloan, Bridge Engineer, Baltimore and Ohio Railroad.

In locations where the cost of concrete is high it may be more economical to use the cantilever or buttressed type of abutment rather than the gravity type. Figure 16-3*e* illustrates a typical cantilever type of abutment as used by the Pennsylvania Turnpike Commission for reinforced-concrete T-beam bridges. Figure 16-3*b* is a half-tone picture of this same abutment, showing the simple but effective ornamentation.

16-4. U-abutments. The U-abutment is a special form of wing-wall abutment in which the wings are parallel with the roadway as shown in Fig. 16-4*a*. Its use is especially suitable where rock slopes make possible stepping up the wing-wall footings to save masonry as shown in Fig. 16-4*b*. This type should not be used in

streams subject to flooding, since there is a large amount of embankment outside of the wings that is not protected against scour. Furthermore there is an abrupt change in the shape and area of the cross section of the stream, which results in creating bad eddying conditions and loss of head.

On account of the wings being partially buried, the unbalanced earth pressure is less than where the wings are not deflected through so large an angle. However, as the magnitude of the earth pressure on the outside of the wall is uncertain, it is customary to design the wings for the full earth pressure of the material on the inside. Where the wing walls are well tied to the breast wall with reinforcing

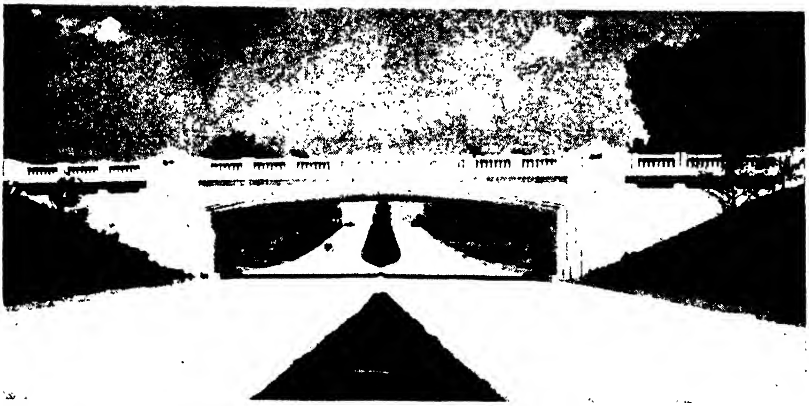


FIG. 16-4*a*.—Bridge Showing U-type Abutments. (*Courtesy of Connecticut State Highway Department.*)

bars, the stability of both the front wall and the wing walls is considerably increased.

Figure 16-4*c* indicates a form of U-abutment in which the side walls are connected by transverse walls and therefore act as continuous horizontal beams of spans equal to the distance between transverse walls. The abutment is floored to distribute the load over a large area of bearing. The side walls are tied into the breast wall with 29 $\frac{3}{4}$-in. rods, and the transverse walls each have 26 $\frac{3}{4}$-in. rods running through them. Openings at the bottom of the transverse walls were provided for the convenience of the workmen.

Figure 16-4*d* illustrates a typical U-abutment as used for highway steel bridges, this design being a former standard of the Ontario Department of Public Highways. The width of the bridge seat and height of backwall will vary somewhat with the type and length of span. For all heights the thickness of the stem at the neat line is made not less than 0.4 of the height of the abutment above the

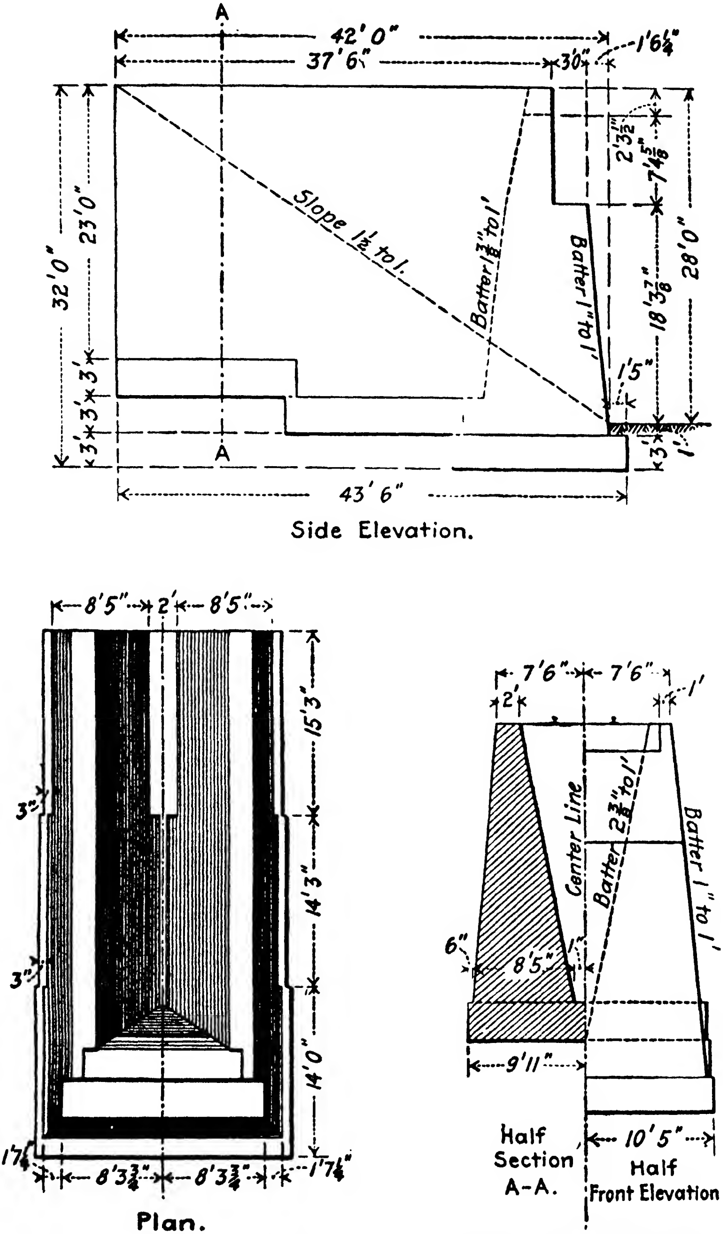


Fig. 16-4b.—Typical Plain Concrete U-abutment, Chicago, Milwaukee and St. Paul Railway.

neat line, this thickness being obtained where necessary, by a series of steps along the back face. The length of wing walls and the

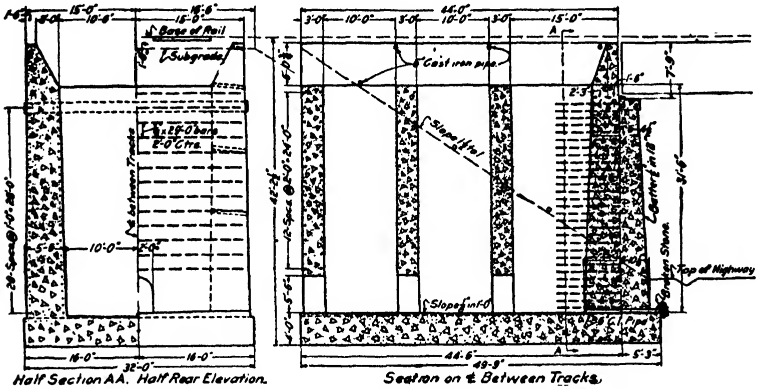


FIG. 16-4c.—Typical U-abutment for Short Plate Girder Spans, Milwaukee, Sparta, and Northwestern Railway.

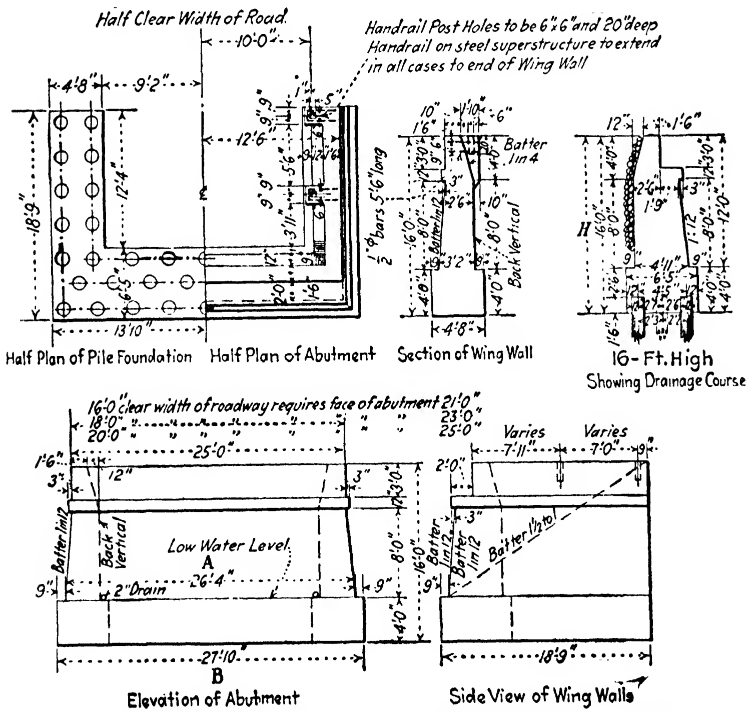


FIG. 16-4d.—Standard Abutment, Ontario Department of Public Highways.

values to be added to the top width of the face wall to get dimensions *A* and *B* are given in the following table, all distances being in feet

and inches. The yardage of concrete for an 18-ft. roadway is also given.

Height H, feet	10	12	14	16	18	20	22
Length of wing wall	9-9	12-9	15-9	18-9	21-9	24-9	27-9
Add to top width for A	0-4	0-8	1-0	1-4	1-8	2-0	2-4
Add to top width for B	1-10	2-2	2-6	2-10	3-2	3-6	3-10
Concrete, cubic yards	50	69	92	117	147	185	230

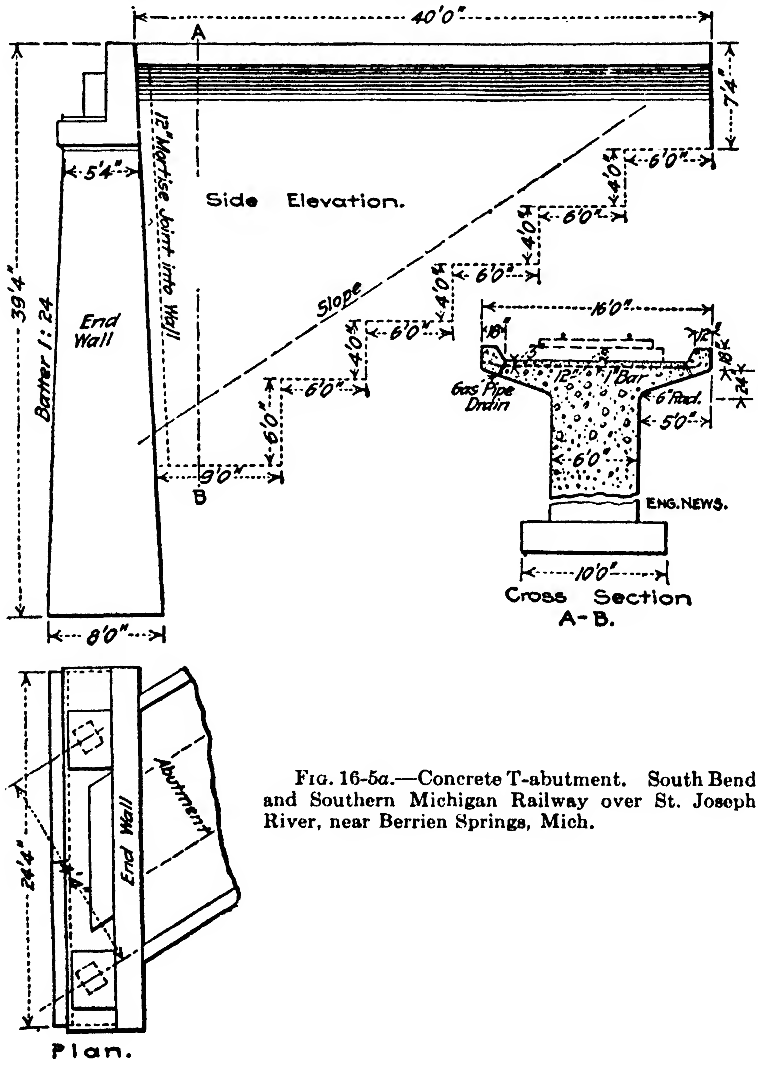


FIG. 16-5a.—Concrete T-abutment. South Bend and Southern Michigan Railway over St. Joseph River, near Berrien Springs, Mich.

FIG. 16-5b.—View of Single-track Railroad Bridge Showing T-abutments.

16-5. T-abutments. The T-abutment, which consists of a front or breast wall for the support of the superstructure and a stem acting as a roadway, was widely used in the early period of railroad construction. Its application is limited to single-track bridges, and since 1880 its use in this field has gradually decreased. Figures 16-5*a* and 16-5*b* are shown chiefly because of the historic interest of this type. The first illustration shows a skew abutment resting on hardpan, which permitted stepping up the base of the stem.

16-6. Buried Abutments. Instead of placing the abutment at the edge of the stream, it is sometimes set back in the embankment

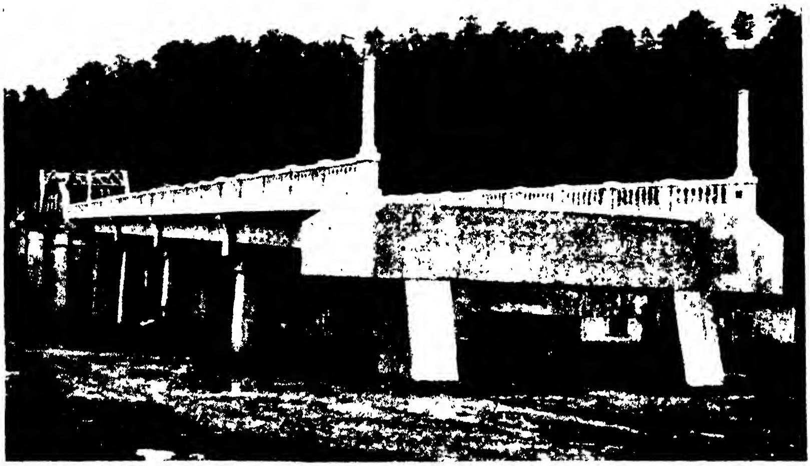


FIG. 16-6*a*.—Showing Buried Type of Abutment. (*Courtesy of L. L. Jemison, Bridge Engineer, State Road Commission of West Virginia.*)

and the latter allowed to spill out in front up to the bridge seat. In this case most of the earth pressure back of the abutment will be balanced by that in front, and hence a much less massive structure is required. On the other hand, a greater length of superstructure is needed. Sometimes a pier is placed at the toe of the embankment, in which case the pier, the buried abutment, and a short span take the place of the regular abutment.

Figure 16-6*a* shows a buried abutment of low height before the approach fill was placed. The breast wall, of shallow depth, rests on two battered shafts.

A higher buried abutment is shown in Fig. 16-6*b*. This was used on the East Haddam bridge across the Connecticut River at East Haddam, Conn., and it consists of two reinforced-concrete columns, two footings, and a transverse slab. The footings are tied together

with four 1-in. rods encased in concrete. The transverse reinforced-concrete slab, 15 in. thick and 6 ft. deep, connects the tops of the columns and keeps the earth filling from the bridge seats, which rest on the columns. A stone slope pavement protects the toe of the filling, which is carried up around the columns to within 6 in. of the bridge seats.

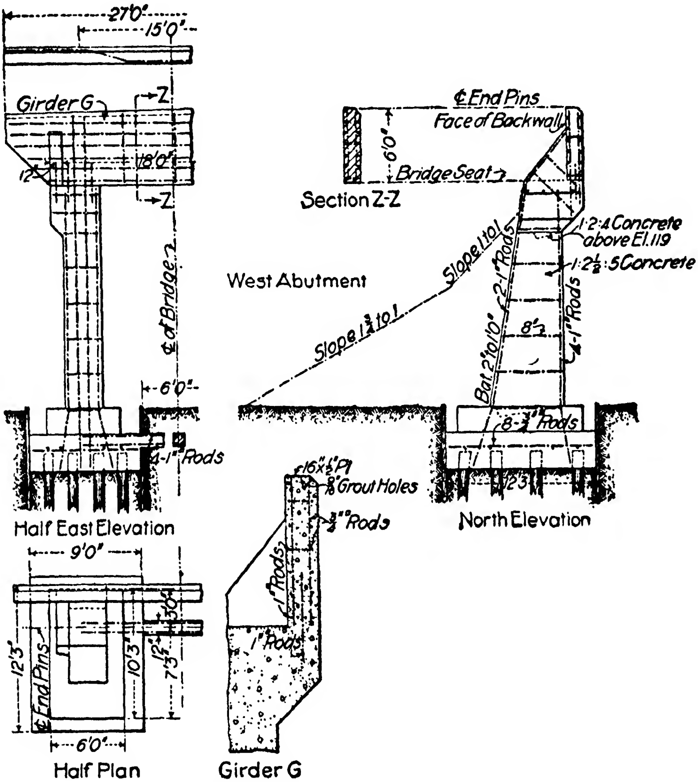


FIG. 16-6b.—A Buried Abutment.

The buried abutment for a bridge on the Louisville and Nashville Railroad, as illustrated in Fig. 16-6c, is an example of a high abutment of this type. It consists of two shafts, a curtain wall connecting them, a spread footing, bridge seats, parapet wall, and side walls. The shafts are 4 ft. thick, 3 ft. 11 in. wide at the top, and 14 ft. wide at the bottom. They are 5 ft. apart in the clear and are connected by a 7-in. curtain wall. Wing buttresses, varying from 6 to 15 in. in thickness and from 0 to 4 ft. 6 in. in width, stiffen the shafts and transfer some of the load to the footing. The toe

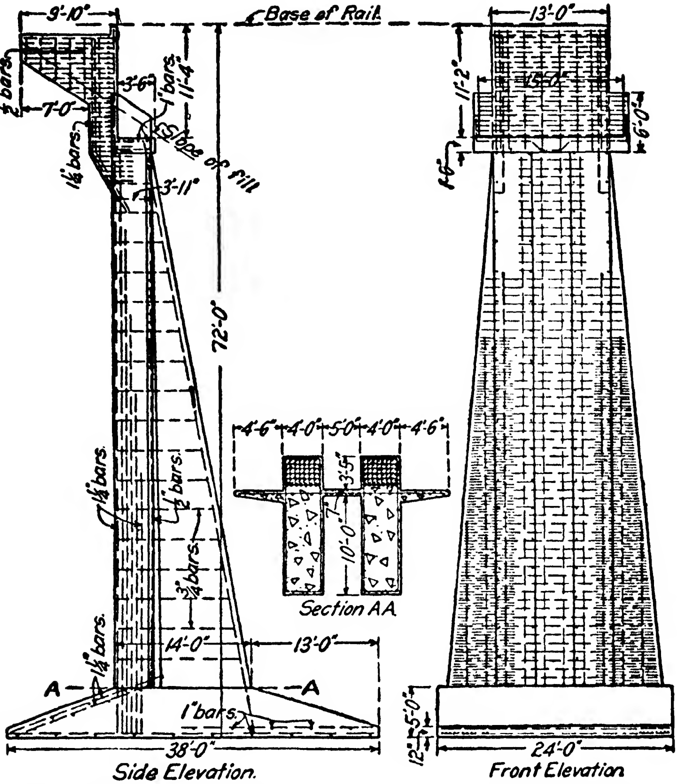


FIG. 16-6c.—Reinforced-concrete Buried Abutment, Louisville and Nashville Railroad Bridge over Cumberland River.

FIG. 16-7a.—Bridge Showing Box-type Abutments. (*Courtesy of F. P. Turner, Principal Assistant Engineer, Norfolk & Western Railway.*)

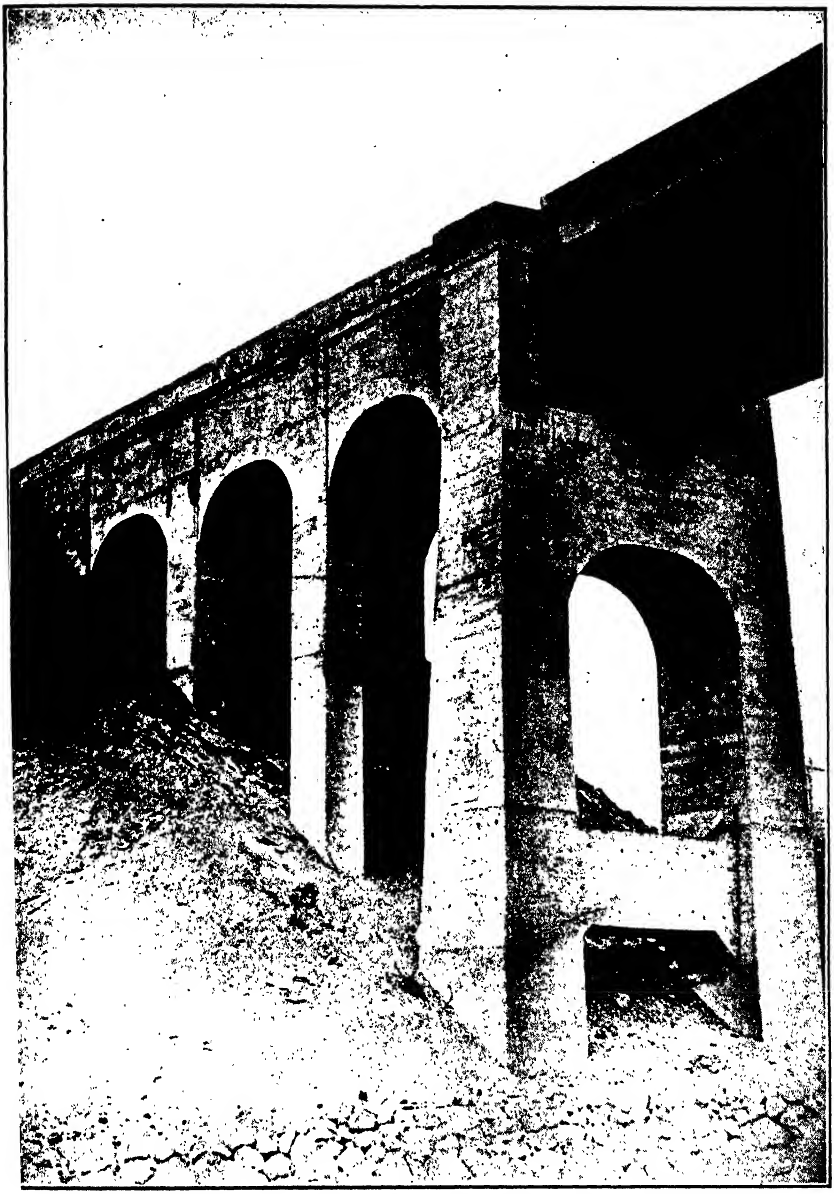


FIG. 16-7b.—Reinforced-concrete Arch Abutment of Lind Viaduct, Chicago, Milwaukee, and St. Paul Railway. Built in 1909.

end of the footing is reinforced near the lower surface, and the heel end is reinforced near both the lower and upper surfaces. The parapet and side walls are each 13 in. thick. The chief function of the side walls is to start the embankment slope far enough back to clear the bridge seats.

16-7. Box-type Abutments. A type of abutment which has come into extensive use in recent years where a railroad crosses a highway having sidewalks is illustrated in Fig. 16-7*a*. In this open box type the superstructure reactions are taken down through the sidewalk columns. The backwall is at the end of the steel span, the approach fill extending beyond the plane of the wing walls and resting on the floor of the abutment. This is an excellent type of design for taking care of the earth-pressure forces in an economical manner. It also has advantages over the type of construction that uses a short steel span resting on columns at the outer edge of the sidewalk and on a regular abutment at the inner edge, in that in the latter type the deflection of the long span over the roadway under live loads tends to raise the abutment end of the short span, resulting in poor riding characteristics and increased maintenance costs.

Another form of the box type of abutment is illustrated in Fig. 16-7*b*, an abutment used on the Lind Viaduct of the Chicago, Milwaukee and St. Paul Railway. The shafts rest on independent concrete footings and are braced with longitudinal and transverse members. The floor system is composed of slabs and beams, the latter running transversely and carrying their loads to the opposite pairs of shafts and to the longitudinal arched beams at a point midway between the shafts, thus making the beam spacing 8 ft. center to center, the shaft spacing longitudinally being 16 ft. center to center.

CHAPTER XVII

UNDERPINNING BUILDINGS

17-1. General. The technical term "underpinning" is used to denote the placing of new foundations or supports under existing structures. As an engineering art and science applied to heavy buildings, this work has been developed almost entirely in a few large cities, notably New York, Chicago, and Boston. In New York, the subways and the modern skyscraper, with its foundations carried far below those of adjacent buildings, have necessitated the placing of new and deeper foundations for many of the older buildings. Some of these underpinned buildings have wall loads as high as 45 tons per lin. ft. and column loads of 1,300 tons or more. The underpinning of such heavy buildings requires great skill and care, for it must be done without resulting in any differential settlement to cause cracking of walls or trouble in operating elevators and other mechanical equipment. Moreover the work must be done rapidly and in a limited space. For an excellent book on underpinning, which treats the subject much more exhaustively than is possible in a single chapter, the reader is referred to "Underpinning," by Prentis and White.

Underpinning is also sometimes necessary due to faulty foundation construction or to changes in soil conditions. In the case of one building the material at some distance below the footings was found to consist of a thick bed of peat. However, this fact was discovered only after the building had settled 6 or 7 in. A change in the elevation of the natural ground-water level often causes trouble. A rise may soften the bearing material, while a lowering may result in the decay of the tops of wooden piling (see Art. 6-2). Settlement also sometimes results from a compaction of the soil when vibrating machinery is installed in the neighborhood.

Underpinning methods may be divided into two general types, (*a*) the pit method and (*b*) the sectional steel-cylinder or caisson method. In the first method, usually used where the new foundations are not to be carried very deep, pits are dug under the existing foundation and the footings carried to a lower level, needle beams being commonly used to temporarily support the structure. In

the second method cylinders or caissons are placed under the existing footings and sunk to rock. Ordinarily needle beams are not needed here since only a small area of support is lost at any one time. Under some conditions a combination of the two methods is used.

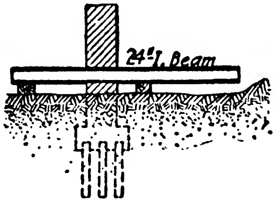


FIG. 17-1*a*.—First Step.

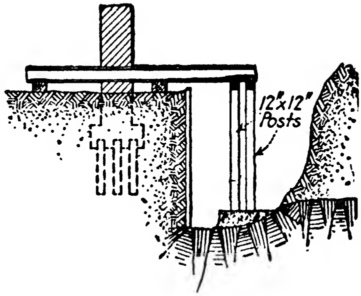


FIG. 17-1*b*.—Second Step.

In transferring wall loads to needle beams, holes are first cut through the walls at intervals of from 3 to 10 ft. or more, depending somewhat on the strength of the walls, after which steel or wooden beams are placed through the openings. The ends of the beams are held on temporary supports placed at a sufficient distance from the wall to permit excavation and reconstruction work to be carried

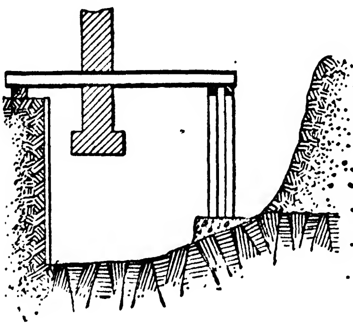


FIG. 17-1*c*.—Third Step.

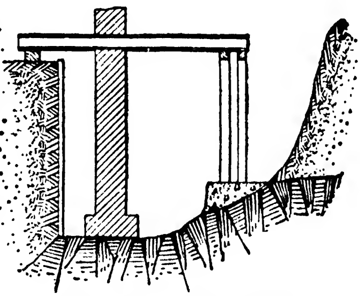


FIG. 17-1*d*.—Fourth Step.

on under the wall. The needles are raised to take the wall load by the use of jacks placed under the ends of the beams.

Figures 17-1*a* to 17-1*d*, inclusive, illustrate the use of needle beams as applied to the Cross Building, New York City. The first step was to cut holes at 6-ft. intervals in the 56-in. walls large enough to admit needle beams consisting of four 24-in. I-beams. The needles were loosely supported on the inside of the old building by blocks placed on the concrete cellar floor and on the outside by

blocks supported on the earth immediately alongside the wall. Sheathing was then driven outside of the blocks, after which excavation was carried to rock. A rough concrete footing was placed on the rock and 12- by 12-in. posts erected to carry the outside ends of the needle beams. Shims were then driven in under the brick wall to transfer the wall load to the needle beams. Sheathing driven on the inside of the old building permitted excavation to be made under the brick walls to rock bottom and the entire old footing removed. The new footing was then built.

The most difficult problem in designing a needle-beam system lies in estimating the load on any particular needle beam. The rest of the design is a matter of elementary mechanics. The total weight of the structure to be supported can usually be approximated with sufficient accuracy. If the needle beams are spaced at equal distances, it will ordinarily be assumed that all take equal loads. Where hydraulic jacks equipped with pressure gages are used, any desired distribution of load can be attained.

17-2. Needle Beams. Needle beams are usually supported (*a*) on struts resting on concrete bases as shown in the illustrations

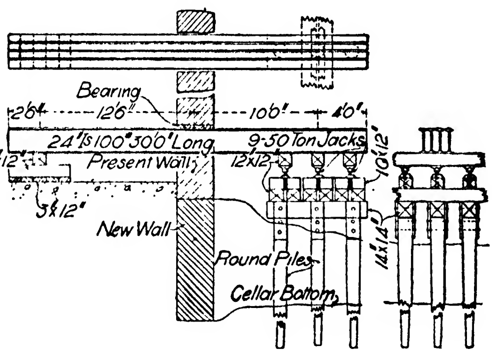


FIG. 17-2*a*.—Underpinning with Needle-beams and Pile Bents.

of the preceding article, (*b*) on piles, or (*c*) on cribbing built on the surface of the ground. The first method is used where good bearing is available near the surface. Where the ground is soft, a pile foundation should be used. The crib form may be used where the loads must be distributed over a considerable surface of the ground.

Figure 17-2*a* shows the details used for underpinning buildings adjacent to the Adams Express Building, New York City, and illustrates the use of piling and of cribbing. Here the inside ends of the needles were supported on blocking resting on the cellar floor, while

the outside ends rested on 12- by 12-in. timbers running parallel to the wall, under which were the 50-ton jacks used in raising and supporting the wall. These in turn rested on small blocks which took bearing on longitudinal 12- by 12-in. timbers, the latter resting on pile bents.

Figure 17-2*b* illustrates the use of needle beams resting on cribwork to support a 300-ton column. The first-floor beam and

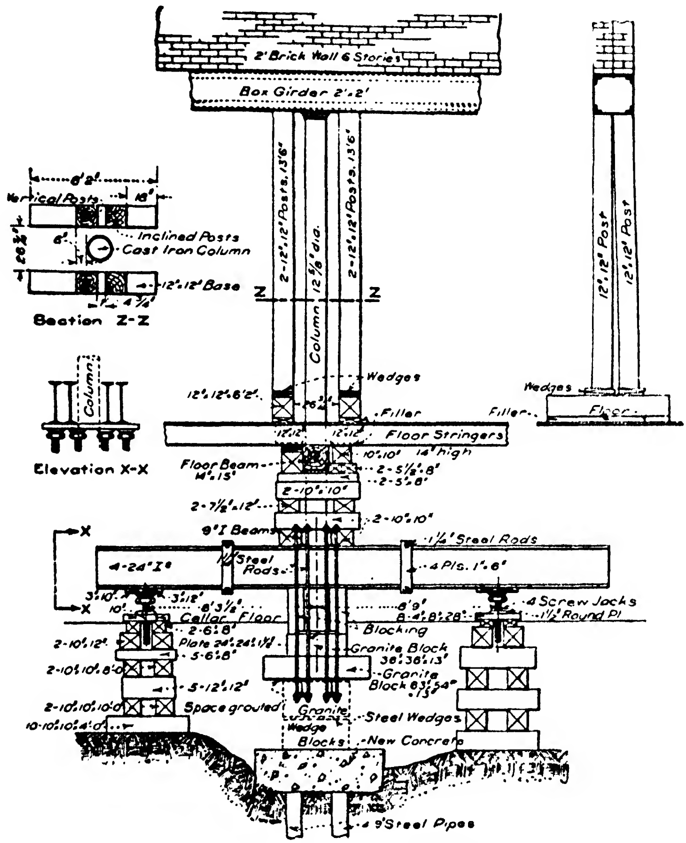


FIG. 17-2*b*.—Underpinning a 300-ton Column on Quicksand, Sargent Building, New York City.

stringers were blocked and wedged up on the needle beams close to the column. Sills were then laid on the floor adjacent to the column to receive two pairs of posts wedged to bearing on the underside of the box girder close to the column. The jacks were then operated to take the floor and wall loads from the column to the cribbing.

Figure 17-3*a* shows the method used in underpinning the Benedict Building in New York City. The needle beams rested on struts on the outside and on cribbing on the inside. Holes about 5 ft. apart were first cut in the wall and into these holes were inserted 15-in. I-beams in groups of three, each group being tied together with iron yokes at both ends. On the outer end two 20-ton jacks carried the reaction into two 12- by 12-in. vertical posts supported by 5- by 5-ft. grillages which rested on a concrete footing.

Another illustration of crib support is shown in Fig. 17-2c. Horizontal I-beams recessed into the face of the wall carried the

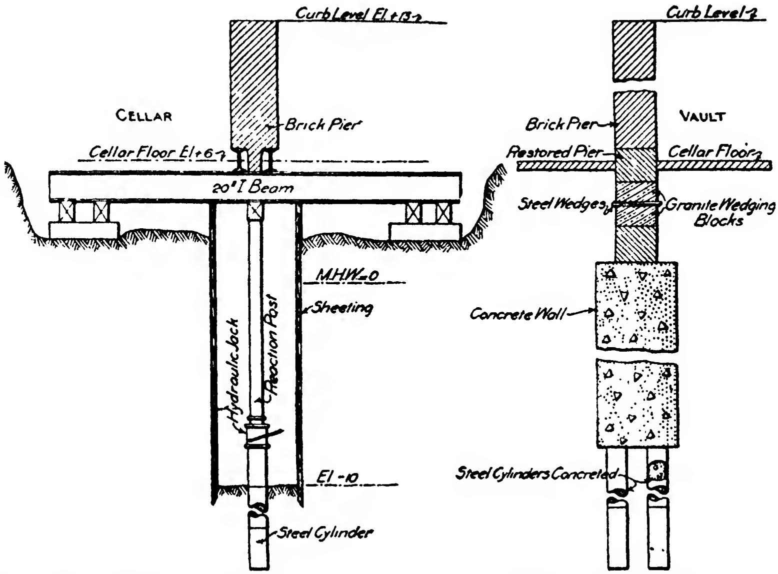


FIG. 17-2c.—Method of Underpinning Centre Street Buildings, New York, Due to Subway Excavations.

weight of the wall to the 20-in. needle beams, which served also to furnish a reaction for sinking sectional steel pipe by hydraulic jacks.

17-3. Supporting Wall below Main Needles. With the needle-beam method of underpinning it is usually impracticable to place the main needles below the old foundation. For this reason, if the new foundation is to be constructed only up to the old one, it becomes necessary to use some special method of supporting the wall and the old foundation below the main needles.

Figure 17-3*a* illustrates one method of doing this. After placing the main needles and their supports, a narrow excavation was made between the old wall and the sheeted pit, the latter being braced

against the face of the masonry as the excavation proceeded. This trench was braced as the soil was removed, and, when the excavation reached the bottom of the old footing, a small drift was extended under it. Springing needles were then inserted to take the load from the wall to the main needles through vertical chains with

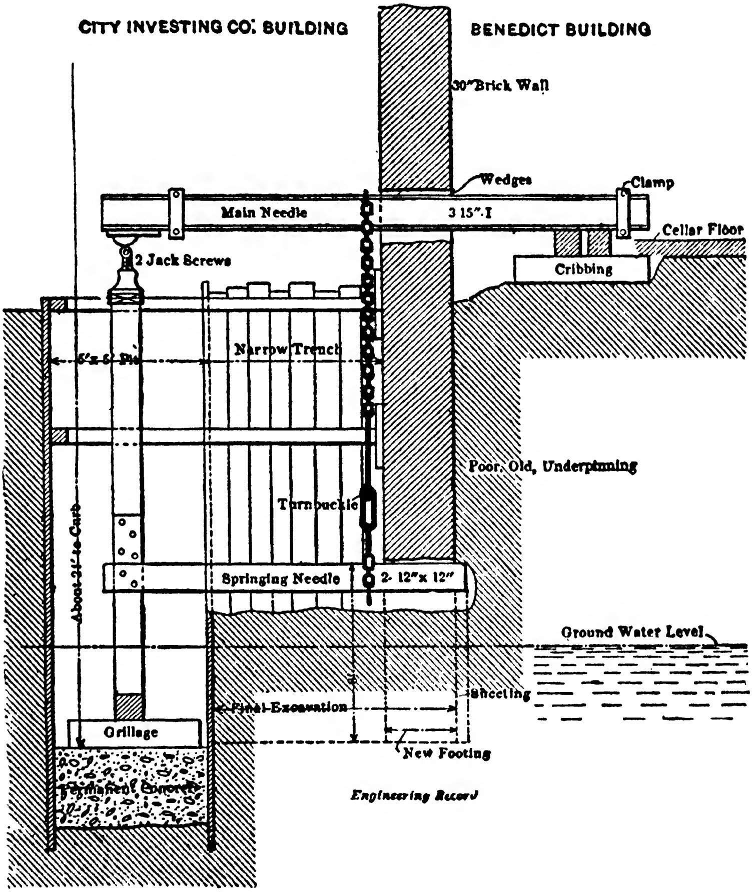


FIG. 17-3*a*.—Underpinning Methods for Benedict Building, New York.

turnbuckle attachments close to the wall. The left ends of the springing needles reacted against a vertical strut (not shown) placed between the springing needles and the main needles to relieve the connection to the vertical shores.

Figure 17-3*b* illustrates a second method for a suspended support, in which a steel bearing plate was seated across the top flanges of the needle beams to give bearing for the nuts in the upper end of two

2-in. vertical rods about 7 ft. long. The nuts on the lower end of the rods engaged a cross plate or saddle, forming a fulcrum for an 8-in. horizontal I-beam 10 ft. long. The long arm of the cantilever reacted against some of the I-beams supporting the outer end of the needle beams. The short arm took bearing about 2 ft. long on the underside of the old concrete footing, supporting it across the thickness of the wall.

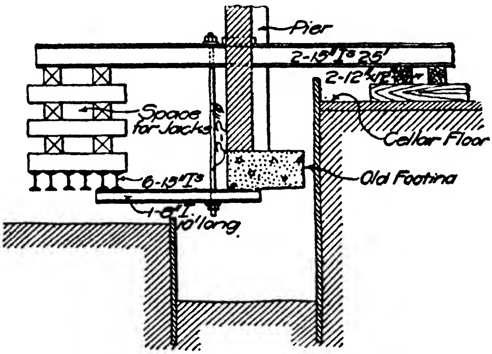


FIG. 17-3*b*.—Suspended Support for Footing, Silversmith's Building, New York.

17-4. The Cantilever Method. Oftentimes conditions make it impossible to occupy the space on both sides of the wall; that on the outside perhaps being taken up with other construction work or that on the inside being used for business purposes. A number of arrangements may be employed to avoid occupying space outside the wall or to decrease the amount, the cantilever method being one of the most popular. Two examples of this method are given to show the fundamental principles.

In the construction of a building at 42 Broadway, New York City, it was necessary to sink caissons close to the seven-story building then occupying the adjacent site, which precluded using the usual type of needle-beaming in underpinning the wall of the seven-story building. The method used consisted in placing two groups of five 20-in. I-beams 30 ft. long and 21 ft. apart in the clear—this distance being necessary to clear the caisson—through the foot of the wall, as shown in Fig. 17-4*a*. These needles rested on cribbing. Four 24-in. I-beams 30 ft. long were then placed parallel with, and close to, the wall and suspended from the needles by yokes. These beams served as a fulcrum to support three sets of four 20-in. cantilever I-beams each. These cantilevers were located as shown in the illustration, converging at one end where a platform was built and loaded with pig iron to form a counter-

weight against the upward reaction. The wall was supported on the needle girders and on the ends of the cantilevers through double rows of 20-ton jacks.

Figure 17-4*b* shows an installation in which all the underpinning was done from the inside. The needle beams, with one end inserted in holes in the wall, were fulcrumed on jacks 5 ft. from the inside face of the wall. The beams took bearing on blocks of wood which were bored at each end for a 4-in. jackscrew and which rested on cast-steel nuts engaging the screws. The lower end of the screws

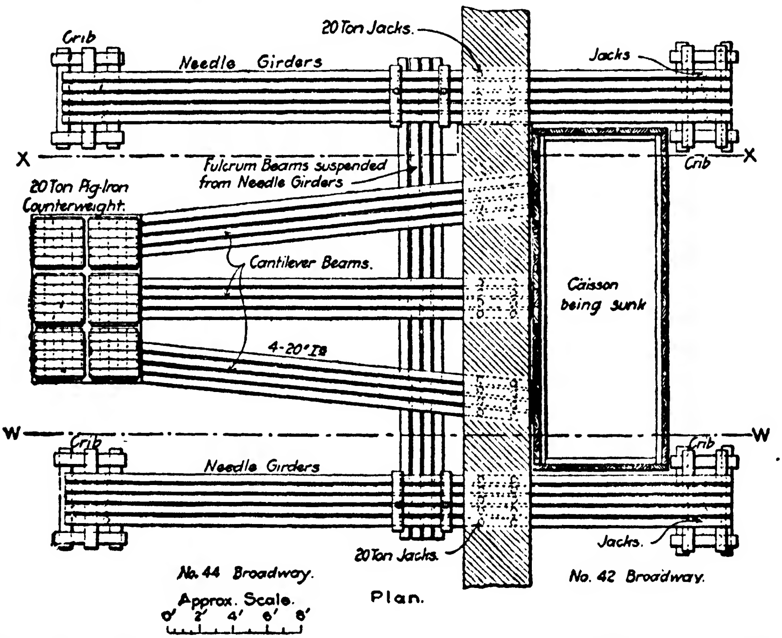


FIG. 17-4*a*.—Counterweighted Needle-beams and Girders for Building at 44 Broadway, New York City.

took bearing on cast-steel base plates seated on sills which transmitted the load to a timber grillage. Auxiliary supports were wedged up against the needles to take the load in case of failure of the jacks.

The wall loads were transferred to the needles through timber blocks with a few inches of cement on top to develop more uniform bearing. The sets of needles were spaced 9 or 10 ft. apart, but, as shown in section *WW*, intermediate bearing was obtained through blocking and wedges resting on 8- by 8-in. horizontal beams. Supporting timbers were jacked up under the lower flanges of the

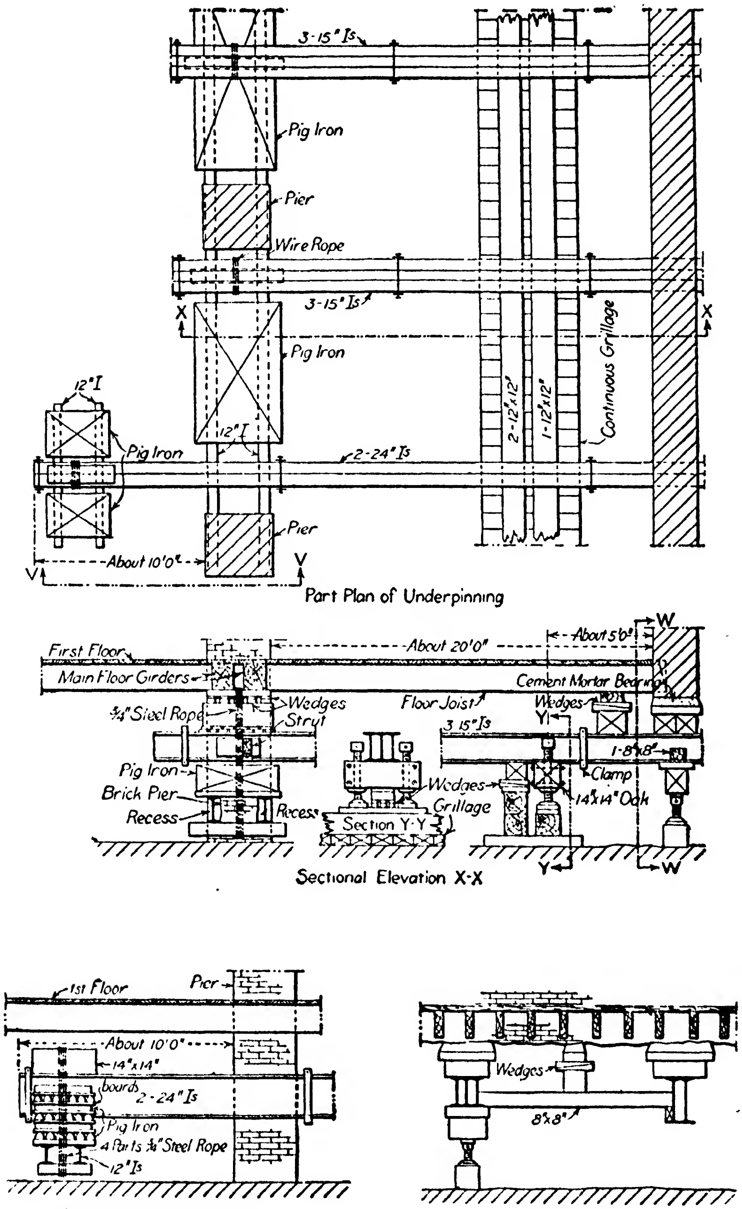


FIG. 17-4b.—Arrangement of Underpinning, 92–94 Maiden Lane, New York City.

needles in the plane of the wall until the new foundation was ready to be built.

The long arms of the cantilevers reacted against the main floor girders in the first floor through blocking and wedges and were further held down by cast-iron ballast and by anchoring to the piers through pairs of horizontal I-beams. The latter engaged recesses in the piers and had transverse pieces across their lower flanges. To these transverse pieces were attached lengths of wire rope passing up over the blocks on top of the needles.

17-5. Figure-4 Needles and Shores. Where working space is not available within the building, the figure-4 method of support may be used. Figure 17-5*a* shows the details of this method as used in the Benedict Building, New York City. Pits 5 ft. square and about 6 ft. on centers were first excavated and sheeted to about 30 ft. below the curb, and on the bottom a 3-ft. layer of concrete was placed. On this concrete a timber grillage was erected to distribute the load from a 12- by 12-in. shore to the concrete footing. The lower end of the 30-ft. shore took bearing against a short horizontal timber, the latter in turn bearing against either one or two jackscrews reacting against foot blocks. The upper end of the shore was surmounted with a saddle plate and wedges and was notched into the wall. The saddle plate gave bearing to 1-in. vertical rod suspenders, to the lower end of which were fastened turnbuckles and chains, engaging the 12- by 12-in. springing needles.

The springing needles which took the wall load by a cantilever action were placed after first excavating the space between the sheeted pits and the wall. The outer ends were bolted to the inclined shores and took bearing against reaction cleats above.

To take the wall load from the old footing, the jacks were first operated to bring the shore to a tight bearing at the top and at the same time the turnbuckles were screwed up to take care of the wall load below the top of shore.

With this type of underpinning, a strong footing must be provided under the shore, for the entire load of the wall is taken by the same. To take care of the horizontal force at the top of the strut, the shore should be inserted at a floor level.

As an auxiliary to other methods shores are widely used in underpinning work. Fundamentally they are figure-4 needle beams without the vertical rod and horizontal springing needles, and as a result they can take only the weight of the wall above the elevation of top of shores. For this reason the use of shores must be combined with some other type of support or else enough of the original

footing must temporarily be left in place to take the weight of the wall below the shores. Figures 17-5*b* and 17-5*c* illustrate the use of shores.

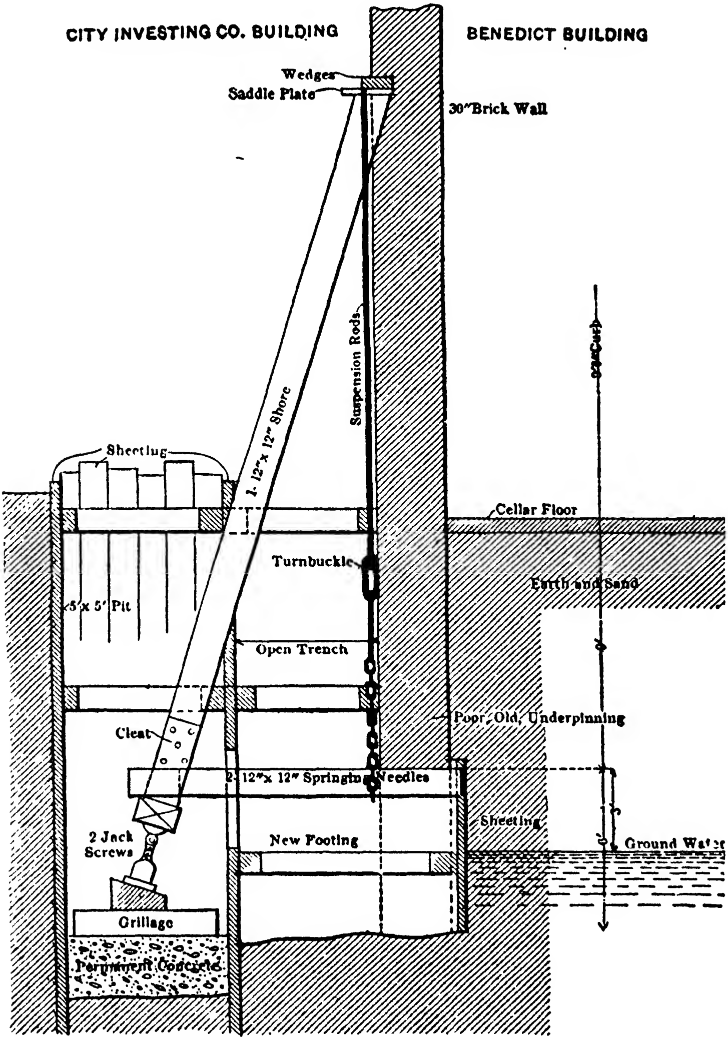


FIG. 17-5*a*.—Underpinning with Figure-4 Needle Method, Benedict Building, New York City.

17-6. Pit Underpinning. In underpinning bearing walls where the loads are only moderately large and the new footing is not very deep, preliminary needling may not be necessary, in which case the work is carried on by the simultaneous digging of a number of pits

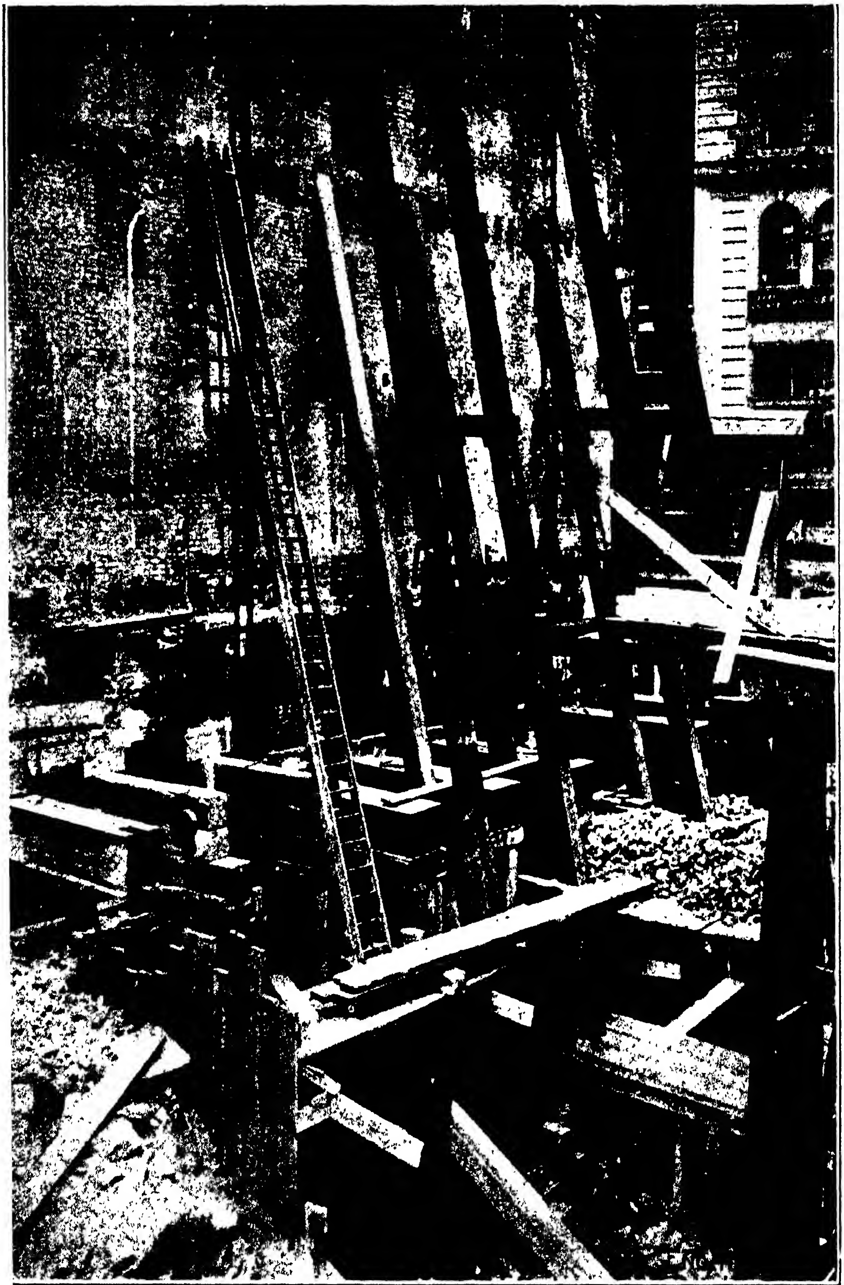


Fig. 17-5b.—Use of Long Shores in Underpinning Cross Building, New York.

at widely separated points. The new footings are then built in these pits to within a short distance of the old footing, and later a

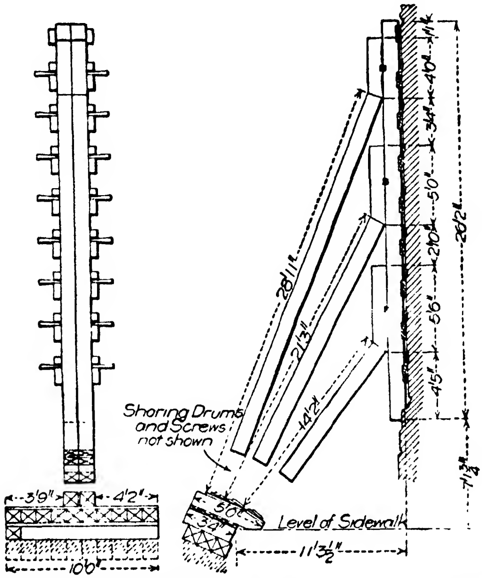


FIG. 17-5c.—Showing Ashlar Face Wall of the Farmers Loan & Trust Co.'s Building in New York City.

connection is made. This is followed by digging more pits and the process is repeated. The finished underpinning may be of the interrupted-pier type in which the wall of the building assumes an arch action between piers; or the underpinning walls may be continuous. The choice will depend on the loads to be carried, the bearing capacity of the soil, and the strength of the building walls. Where necessary, the new footing may be spread at the base.

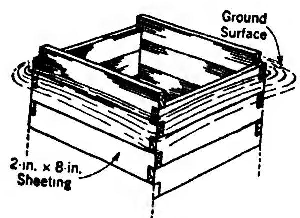


FIG. 17-6a.—Method of Dovetailing Sheeting for Underpinning Pits.

Underpinning pits are usually about 4 or 5 ft. square and lined with horizontal sheeting. Figure 17-6a illustrates a patented type of dovetail sheeting used in underpinning work for subway construction in Brooklyn, N. Y.

The normal procedure was to dig down along one side of the pit a distance of 8 in. below the bottom board and smooth off the damp sand so that there was a clear vertical face of sand 8 in. by the width of the pit. A notched board was set in place against this vertical face and then forced

snugly against the sand by being tapped with a hammer to ensure a close fit. The opposite board was put in similarly, and after that the boards on the two remaining sides were gently forced into place, making a snug fit. By this method pits as large as 6 by 6 ft. were sunk through damp sand 65 ft.[1]

These pits were put down at a total cost of labor and materials per foot of depth of from $4 to $5. Where the building is thoroughly needle-beamed, the pits may be excavated in longer units and less care need be exercised in placing the lining.

For depths up to 15 ft. the earth is usually removed by shoveling in stages, using scaffolds, but for greater depths it is more economical to use buckets and winches. Where the pits extend below ground-water elevation, it is generally necessary to use interlocking vertical sheet piling, driven down well in advance of excavation and preferably to an impervious stratum. Where the flow of water through the bottom cannot be controlled by a reasonable amount of pumping, then some other method, such as steel piles, open caissons, or pneumatic caissons must be used.

17-7. Joining to the Old Wall. One method used to join the new wall to the old wall is to bring the concrete to within a few inches of the old concrete, and then, after the new concrete has hardened, to dry-pack the space left with a stiff mortar. This is rammed tightly into the open space by any convenient means such as using a 2- by 4-in. piece of timber and a fairly heavy hammer.

The use of quick-setting cements has made possible the expediting of underpinning work, for now a new pit can be started within a few hours after the preceding pit has been concreted and dry-packed.

Where the dry-pack method is used, some settlement will invariably result. A more effective method of load transfer is illustrated in Fig. 17-2*c* (right-hand figure). On this job, when the new concrete foundation was placed, brick piers were built upon the new foundation between the needles to within a few feet of the bottom of the old wall. On these piers were placed pairs of cut stones of the same length and thickness as the piers, and about 14 in. high. One stone set loosely on the other, with pairs of steel wedges between. The brick pier was then continued on the upper stones until the underside of the old wall was reached, to which it was carefully joined. The wedges were driven together until the load was lifted from the needles, after which the latter were removed and the brickwork of the wall completed. The wedges were then

[1] *Civil Eng.*, vol. 4, p. 367, July, 1934.

sawed off flush with the face of the wall and the space between the stones filled with grout. Theoretically the entire load was carried through the wedges, but actually some settlement doubtless took place to distribute the load throughout the length of the wall.

17-8. Steel-cylinder Underpinning. The use of sectional metal pipes filled with concrete for underpinning buildings was developed and patented by Jules Breuchaud[1] and first used in underpinning buildings in New York City about 1896. In underpinning a wall a typical procedure is to cut a horizontal and a vertical recess in the wall in the form of a T. One or more I-beams are set in the horizontal recess, after which the first section of pipe is set in the

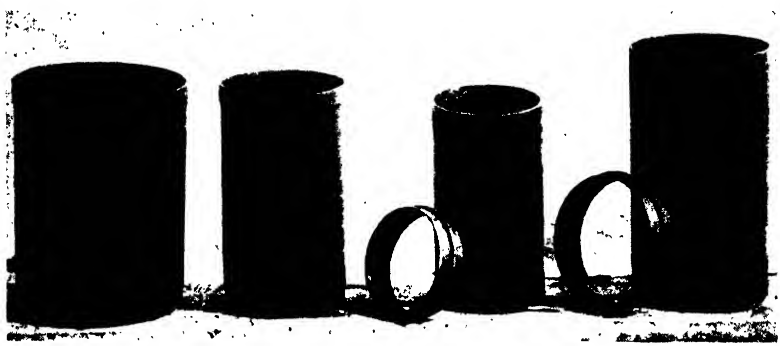


FIG. 17-8a.—Sections of Pipe and Sleeve Connections. (*Taken from Underpinning, by Prentis and White.*)

vertical recess and sunk by jacking against the beams. When this section is sunk and excavated, more sections are added and the operation repeated. On the completion of the sinking, the pipe is filled with concrete and wedged against the beams to take the wall load. The pipes are usually spaced from 5 to 12 ft. apart.

By using cylinders, the wall is deprived of only a small amount of support at any one time; hence needle beams are not generally required. This results in the following advantages: (*a*) less space is occupied; (*b*) the basement of the building need not be entered, the cutting being done from the outside and often not entirely through the wall; and (*c*) it is cheaper than the pit method if the new foundation has to be carried to a considerable depth.

The early cylinders were of cast iron, but today steel pipe is most generally used. The diameter of the pipe varies from 10 in. to 54 in. or more, while the thickness of the shell varies from $\frac{1}{8}$ to $\frac{3}{4}$ in. or

[1] See *Trans. A.S.C.E.*, vol. 37, p. 31, June, 1897.

more. In the smaller diameters the cylinders are practically identical with the tubular piles described in Arts. 7-8 and 7-9, except that the sections are shorter, being generally from 2 to 4 ft. long. The most commonly used diameters are from 12 to 16 in., and the usual thicknesses are from $\frac{1}{8}$ to $\frac{3}{8}$ in. Figure 17-8*a* shows typical sections of pipe and sleeve connections for the two heavier $\frac{3}{8}$-in.

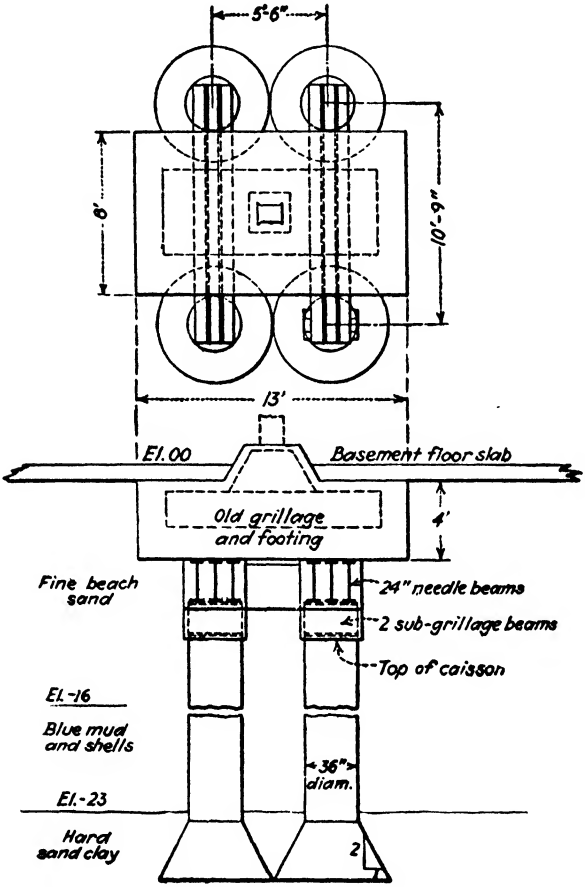


FIG. 17-8*b*.—Underpinning a Column with Four 36-In. Cylinders.

pipes shown on the right. These sleeves are usually of cast steel, although sometimes cast iron is used, and are similar to those described in Art. 7-8. The two thin-shell sections shown on the left of the illustration are reinforced at the top on the outside with a band 2 in. wide and on the bottom with an inside band about 4 in. wide and extending 2 in. below the bottom. These bands furnish the connections between the sections.

Sometimes the use of steel cylinders is combined with the pit method, as shown in Fig. 17-2*c*. Here a pit about 12 ft. deep was first excavated, and the cylinders were sunk from the bottom of the pit.

Where the larger sizes of pipe are used, men can work in them, and they are virtually open caissons (Art. 9-6). These are sometimes sunk under air pressure.

Figure 17-8*b* illustrates the use of 36-in. cylinders to underpin the column footings of a 14-story building in San Francisco. The original footings consisted of steel grillages resting on timber piles, and these were settling due to failure of the piles (see Art. 4-9). The underpinning cylinders, which were placed outside of the old footing, consisted of $2\frac{1}{2}$-ft. lengths of $\frac{3}{8}$-in. butt-welded steel pipe. The joints consisted of a 6-in. collar of No. 10 gage steel welded on the outside of the lower edge of each section. Sinking was effected by jacking against I-beams temporarily placed under the old footings and by removing the material on the inside of the cylinders by hand. The bottoms were belled out to twice the diameter of the pipe, after which the cylinders were filled with 2,500-lb. concrete.

The 610-ton column load, originally carried on 18 piles, was transferred to the four cylinders as follows. Two 3-ft. lengths of 20-in. I-beams were placed on each cylinder with sufficient space between each pair of beams to permit the operation of a hydraulic jack. Enough timber piles were then removed to permit sliding a 24-in. I-beam under the old footing, with its ends resting on the 20-in. subgrillage beams. This beam was then jacked up and the process repeated for the other beams. The new grillage was enclosed in concrete to provide a 4-in. coverage for all the new steel.

17-9. Sinking Cylinders. The cylinders used for underpinning may be sunk (*a*) by driving with drop- or steam-hammers, (*b*) by hydraulic or screw jacks bearing against the old masonry, (*c*) by using a water-jet to loosen the material around the cutting edge, and (*d*) by excavating the material on the inside.

Where headroom permits, as in the case where the cylinders are offset from the wall or column, the smaller sizes are often driven with a 500-lb. drop-hammer falling 2 or 3 ft. and striking on a steel pile cap. Steam-hammers are often used and have been employed in driving cylinders as large as 54 in.

In jacking work a short section of pipe is first set in place, with a heavy steel plate placed on top. A jack is placed on this plate and blocked up against the footing with jacking dice, which are short lengths of 6-in. pipe filled with concrete. After forcing the first

section into the ground, it is cleaned out and another section added, and the process repeated until the pipe reaches hard bearing. Hydraulic jacks are generally used, a popular size having a $4\frac{1}{2}$-in. ram and a stroke of about 11 in. Working pressures up to about 5,000 lb. per sq. in. are used, which for this size of jack gives a lifting capacity of 40 tons. For most satisfactory results the pump and jack are made independent and not as a single tool. The jacks can

FIG. 17-9*a*.—Tools Used in Excavating Soil in Cylinders. (*Taken from Underpinning, by Prentis and White.*)

then be made much lighter, and the pump is more easily and conveniently operated. Power-operated pumps, automatically controlled by hydropneumatic accumulators, are desirable where a considerable amount of jacking is to be done.

Among the tools used for excavating the soil in the cylinders are posthole diggers, earth augers, miniature orange-peel buckets, water-jets and air jets (see Fig. 17-9*a*). The two first-named are used for moderate depths where water is absent. Orange-peel buckets are suitable for digging coarse material, for lifting small boulders, and for general use in water-bearing soil. The water-jet and the air-jet methods have been used very successfully in connection with tubular-pile driving for new foundations (Art. 7-8), but, where used

in underpinning operations, great care must be exercised to prevent wetting the pit and thus softening the ground which supports the old foundation. Care must also be taken to prevent scouring action below the bottom of the cylinder. However, the blowout method as used for the piles of the Starrett-Lehigh Building and described in Art. 7-9 has been successfully applied to underpinning pile work.

The larger cylinders are generally sunk by hydraulic jacking, the material being removed by men working in the cylinders. The muck is sometimes washed out, assisted perhaps by a sand pump.

17-10. Concreting Cylinders. Where small diameters are used, little can be done in preparing the bottom further than to pump out the water and to inspect the bottom by means of an electric light to see if the desired bearing has been reached and all loose material removed. With the larger cylinders the bottom can be cleaned, leveled off, and the bearing area widened if desired.

A 1:2:4 concrete is ordinarily used for the filling. Where there is considerable water present, it is best to place a few feet of richer concrete through the water and, after this has hardened, to pump out the water and place the remainder in the dry. Where the concrete is deposited through water, a cylindrical bucket of a diameter somewhat smaller than that of the pipe and about 3 ft. long is often used. The bucket has a flap bottom and two lines, one attached to the bail of the bucket and the other to the flap. In lowering the bucket, the weight is carried by the flap line, but, after the bucket is seated on the bottom, it is lifted by the bail line, which causes the concrete to be deposited through the bottom. This method largely prevents the water from separating the constituent materials of the concrete.

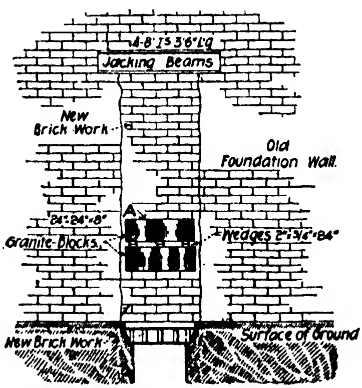


FIG. 17-11*a*.—Underpinning the Empire Building, New York.

17-11. Transferring Loads to Cylinders. Figure 17-11*a* illustrates the method used in transferring the loads to the cylinders of the Empire Building, New York City. The underpinning cylinders were first capped and the recess above them filled with brickwork to a certain height. On this brickwork two granite blocks were placed, one resting loosely on the other, after which the remainder

of the brickwork was placed. Pairs of steel wedges were inserted between the granite bearing blocks and driven together, thus separating the two blocks and bringing the wall loads to the cylinders. The space between the blocks was then filled with cement grout.

In underpinning the Stokes Building in New York City, one of the first in which the pipe method was used, the top of each cylinder

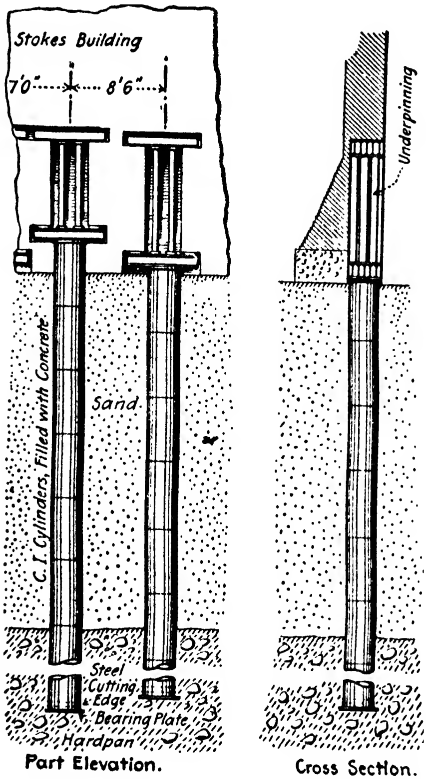


Fig. 17-11*b*.—Underpinning the Stokes Building, New York City.

was capped with a top bearing plate. Five I-beams were placed in a horizontal recess and rested on top of the cylinder, as shown in Fig. 17-11*b*. Steel posts were set in a vertical recess above the horizontal recess, these posts bearing on a steel plate which in turn took bearing on the I-beams below. Another set of I-beams were placed in a horizontal recess above, with a clearance between these beams and the posts of 2 in. Steel plates were placed on the posts and pairs of forged-steel wedges then driven between the plates and the bottom of the upper tier of beams to bring the wall load to

the cylinder. After this the recess was solidly bricked up. Using two tiers of horizontal beams served to better distribute the load out through the brickwork.

An improved method of transferring loads, known as the pretest method, was developed and patented about 1915 by Lazarus White. In using this method each cylinder is loaded by hydraulic jacks bearing against the footing to about 50 per cent more than the design load, the wedges being driven while this load is on. The details of operation differ widely according to the type of job. As a typical example, after sinking a cylinder and leaving sufficient clearance to admit jacks between the cylinder and the underside of the footing, a plate is placed on the pile top to support two jacks. These jacks, placed a sufficient distance apart to later permit the insertion of a short column, are extended to bear against a plate on the underside of the footing. The jacks are then further extended to take the desired load as shown by a pressure gage. A proper length of I-beam placed on end is then inserted between the jacks. After this the necessary filler plates and wedges are installed between the top of the I-beam and the bottom of the plate on the underside of the footing, and the wedges are driven home. The jacks are then unloaded and removed for service on the next cylinder. After all the cylinders have been loaded, the struts are usually enclosed in concrete, this extending down some distance over the cylinder tops.

The pretest method was used in the underpinning work illustrated in Fig. 17-8*b* and described in Art. 17-8. Jacks were placed under the 24-in. needle beams and between the pairs of subgrillage beams and extended until their load was 25 per cent in excess of the design load. Steel shims were driven to provide bearing between the ends of the needle beams and the subgrillage beams, after which the jacks were operated to transfer their loads to the subgrillage beams. Jacking and shimming were done on only one beam at a time.

Pretesting, or preloading as it may properly be designated, serves the double purpose of testing the load-carrying capacity of each cylinder and of eliminating footing settlement which would otherwise be caused by the elastic distortion of the cylinder, since it is never possible to fully load a cylinder by driving wedges alone.

The pretest method has been used in the construction of new buildings as well as in the underpinning of old structures, the piling being carried down as the building is erected, thus effecting a saving of time.

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